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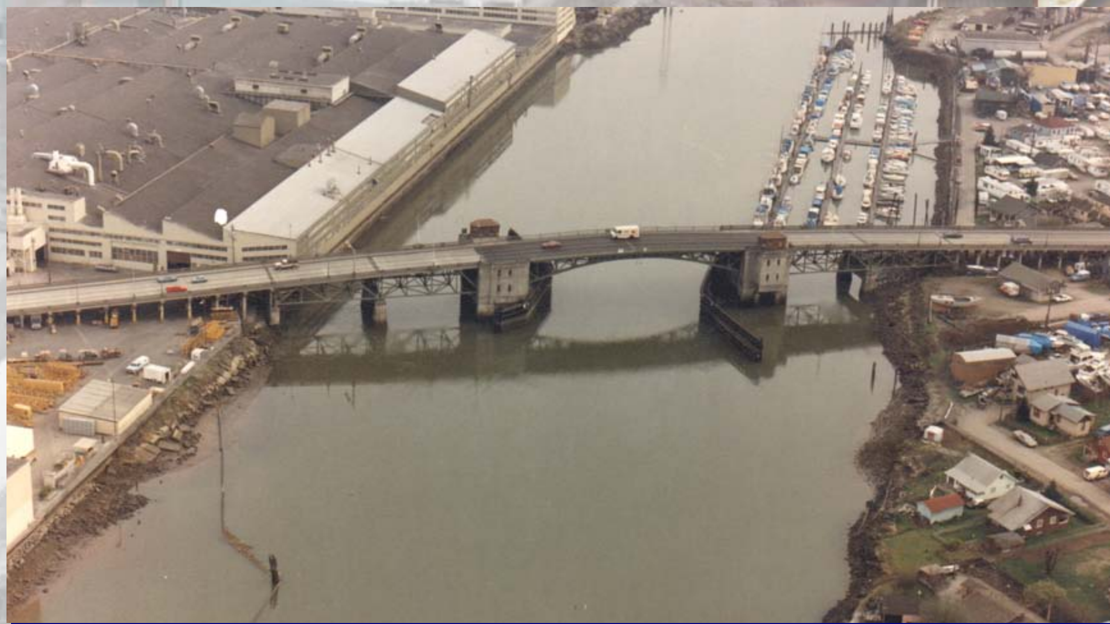

1. EXECUTIVE SUMMARY

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1.1 PROJECT BACKGROUND

King County's efforts to properly maintain the South Park Bridge and keep the bascule spans operational have resulted in numerous bridge inspections and engineering studies. Since 1990, over 30 studies have been conducted that evaluated the soils, analyzed deficiencies, assessed load carrying capacities, and endeavored to determine its remaining life. The objective of this technical memorandum is to provide the County with a single document that contains a peer review of relevant studies, compares the South Park Bridge with similar bridges in the Seattle area, identifies risk factors for the existing bridge, and offers recommendations for next steps.

1.2 PEER REVIEW

The reports reviewed for this memorandum, all conclude the South Park Bridge is in need of major rehabilitation or replacement. While these conclusions are similar, the approach and methodologies which led to these conclusions varied amongst the documents reviewed, due to the complexity and uncertainties associated with the bridge structure.

1.3 BRIDGE COMPARISON

Detailed information from the Seattle Department of Transportation (SDOT) for the Ballard, Fremont, and University Bridges was reviewed for comparison purposes. These bascule bridges were constructed between 1917 and 1919 as compared to 1931 for the South Park Bridge. The South Park Bridge is more vulnerable than the three Seattle area bridges for the following reasons:

- Soil conditions – As described in several of the reports reviewed, the surrounding soils at the South Park site are subject to liquefaction and lateral spreading after a moderate seismic event. The three Seattle bridges are founded in materials less prone to these conditions.
- Bascule pier movement – The South Park Bridge is experiencing movement that is very difficult to define, let alone correct, while the SDOT bridges have not experienced this problem.
- Ongoing concrete degradation – The circumstances creating the concrete degradation throughout the South Park Bridge do not exist on the SDOT bridge structures. In addition the South Park Bridge sits in brackish water while the Seattle bridges are in freshwater.

1.4 RISK ANALYSIS

There are three distinct types of risk that imperil any plan for continued operation of the bridge: economic, operational, and safety. Each type exposes the County to the loss of bridge function and raises concern for public safety.

- Economic risk relates to investing in an aging bridge structure. Based on the cost-based risk analysis, planning should begin immediately for either bridge closure or replacement. The fact that future investment in repairs and/or rehabilitation will likely equal the replacement cost of the structure in approximately 12 years from the year 2005 is reason to immediately begin planning for closure or replacement of the bridge.
- Operational risk is associated with the inability to open or close the bridge. There is a significant risk of the inability to either open and/or close the bridge following a moderate seismic event.

The South Park Bridge has a one in three chance of a damaging earthquake occurring in the next ten years. Should such an event occur, it will likely cause immediate closure of the bascule span, rendering the bridge inoperable.

- The immediate safety risk is the deteriorated condition of the reinforced concrete approach spans. Deterioration of the concrete and reinforcing steel in these portions of the bridge is severe, and both the vertical and lateral load carrying capacity of the approach spans is suspect. It has been estimated that a moderate seismic event will cause significant damage and may lead to structural collapse.

1.5 RECOMMENDATIONS

Based on the reviewed reports and the risk analysis it is evident that the bridge should be replaced. Prior to replacement or closure, the bridge is vulnerable to seismic events, and suffers from continuing deterioration that cannot be repaired. The remaining questions are when should the bridge be replaced and what action should be taken until then.

The bridge is only as strong as its weakest link and that link is the concrete approach spans. The vulnerability of the concrete approaches to seismic events and their continuing deterioration are reasons to replace the bridge immediately. Furthermore, it is not economically, operationally, or structurally reasonable to replace the approaches without replacing the entire bridge structure.

In the interim, the following are recommendations for immediate action:

- Immediately begin planning for replacement and/or closure of the bridge (See Sections 4.2.3, 5.2.3, and 6.3.3.)
- Load test the concrete and steel truss approach structures to determine their load carrying capacity (See Sections 5.3.4 and 4.3.3.) A load test may identify the need to adjust the load rating of the bridge.
- Strengthen the bridge to prevent collapse during a moderate seismic event. Locations to be strengthened include the concrete and steel truss approaches, the bascule piers, and the counterweight.

Following are recommendations for future action:

- Conduct a comprehensive survey of the bridge and establish survey points to be used for ongoing monitoring to assist in determining the operational life of the bascule span.
- Core bascule piers to verify assumptions made in previous analytical reports. The concrete of the bascule piers is suffering advanced deterioration; however, the extent of the deterioration is not known. Previous calculations made to determine the capacity of the bascule piers assume that deterioration is a very small percentage of the overall concrete mass. This needs to be verified.

2. INTRODUCTION

2. INTRODUCTION

2.1 PROJECT BACKGROUND

The South Park Bridge is adjacent to the South Park neighborhood just south of Seattle, Washington. The bridge spans the Duwamish River between unincorporated King County and the City of Tukwila. The bascule bridge was constructed between 1929 and 1931 and is a Scherzer Rolling Lift consisting of two moveable leaves. The bascule span has approaches on each side consisting of two steel truss and twelve concrete flat slab spans.

This vintage bridge was designed to standards that are now outdated. The structure has incurred significant deterioration over its 70 plus years of service and is experiencing unexplained movement that is hindering the ability of the bascules to operate. Coupled with the structure's location in a seismically active region, the County is faced with the need to find a reasonable long-term solution for this Duwamish River span while maintaining the near-term safety of the traveling public.

2.2 SCOPE OF WORK

The scope of work for this memorandum is defined by the following three distinct tasks:

2.2.1 Technical Peer Review

Since the late 1980s, numerous bridge inspections have been conducted and reports written addressing various structural and geotechnical aspects of the bridge. One objective of this memorandum is to provide the County with an independent peer review of the structural and geotechnical content of these reports and identify missing, unclear, or underdeveloped elements and comment on the previous studies' conclusions. King County requested that the following reports be reviewed:

1. *Geotechnical Report, 16th Avenue South Bridge, Seattle, Washington*, Shannon & Wilson, September 1991
2. *14th/16th Avenue South Bridge, Foundation Design Report, North Pier*, Sverdrup, October 1991
3. *14th/16th Avenue South Bridge, Operational Study and Life Cycle Cost Analysis, Comparing Bridge Replacement versus Rehabilitation*, Sverdrup, April 1993
4. *Liquefaction Evaluation, 16th Avenue South Bridge Approaches, Seattle, Washington*, Shannon & Wilson, August 1994
5. *Substructure Investigation, Testing & Assessment of Bascule Piers,* Echelon Engineering, Inc., September 1994
6. *Sixteenth Avenue South Bridge, Concrete Condition Survey, Seattle, Washington,* Boss & Mayes Testing Engineers, Inc., November 1994
7. *14th/16th Avenue South Bridge, Concrete Approach Span, Load Rating Analysis,* Sverdrup, December 1994
8. *Bridge Load Rating & Inspection Memorandum, Steel Approach Trusses and Bascule Span*, Sverdrup Civil, Inc, January 1996
9. *Underwater Inspection Report for 14th/16th Avenue South Bridge,* Echelon Engineering, Inc., November 1997

10. *Seismic Vulnerability Assessment of 14th/16th Avenue South Bridge, Seattle, Washington,* EQE International, February 1998

11. *Seismic Study of 14th Avenue South Bridge,* Imbsen & Associates, Inc. August 2001

12. *South Park Bridge Rehabilitation Feasibility Study*, Parsons Brinckerhoff, Inc, May 2003

13. *Structural Alternatives Study,* Parsons Brinckerhoff, November 2003

14. *Geotechnical Report Phase II, South Park Bridge Project, King County, Washington,* Shannon & Wilson, Inc., March 2004

15. *Analysis of Risk and Remaining Life of the South Park Bridge*, Parsons Brinckerhoff, March 2004

KPFF requested the following additional documentation to assist in the peer review process:

16. *Concrete Condition Survey*, Professional Services Industries, Inc. (PSI), January 2003

17. *Underwater Inspection*, Han-Padron Associates, June 2001

18. *Report of Post-Event Underwater Inspection*, Han-Padron Associates, March 2001

19. *Evaluation of Deteriorated Bridge Substructure*, Han-Padron Associates, March 2001

20. *Underwater Inspection Report*, Collins Engineers, Inc., December 2003

21. *Remaining Service Life Estimate – Subject Area 1, Concrete Condition*, Parson Brinckerhoff Limited, February 2004

22. *Ongoing Bascule Pier Foundation Monitoring*, King County, February 2002

23. *South Park Bridge – Leaf Alignment History,* King Count, November 2005

2.2.2 Vulnerability Comparison

The vulnerability of the South Park Bridge was compared to three other bascule bridges in the Seattle area that carry 20,000 vehicles or more per day. The bridges are the Ballard, Fremont, and University Bridges.

2.2.3 Risk Analysis

The risk analysis identified a comprehensive set of risk factors for the existing bridge. Relationships between level of risk, time, and applicable risk thresholds are explained. In addition, based on the identified risk factors, failure scenarios were studied that would be impractical to repair or lead to bridge closure.

3. SITE CONDITIONS

3. SITE CONDITIONS

A geotechnical review of documents relating to the subsurface conditions at the South Park Bridge was conducted and an analysis of the site-specific acceleration was performed. The results of these tasks are presented in this section.

3.1 SUBSURFACE CONDITIONS

The understanding of the soil conditions is based on borings completed at the site, laboratory and field-testing, and discussions of the pile driving information for the bridge bascules. Borings at the site were drilled in 1964 (north side of bridge), 1991 (north bascule), 1994 (south side of bridge), and 2003 (entire length of site).

3.1.1 Soil Conditions

The subsurface conditions generally consist of fill over recent alluvial and estuarine deposits that are underlain at depth by glacially consolidated soils. The fill generally consists of loose to medium dense, slightly silty to silty sand and was observed up to 12 feet thick. The alluvial deposits generally consist of very soft to medium stiff clayey silt and loose to medium dense slightly silty to silty sand. The alluvial deposits range from approximately 30 to 60 feet thick. The estuarine deposits generally consist of a thin layer (0 to 5 feet thick) of very soft sandy, clayey silt over a thicker layer (5 to 25 feet) of medium dense to very dense, silty, gravelly sand and sandy gravel. The soil explorations terminated at glacially consolidated materials consisting of hard, clayey silt to silty clay with some sand and gravel. Sandstone bedrock has been observed in soil explorations to the south of the project site, but was not observed in the site borings.

3.1.2 Groundwater Conditions

Groundwater is located within 10 to 12 feet of the ground surface throughout most of the site. A study by Shannon & Wilson (referenced in 2004 document) indicated that the Duwamish Waterway in the vicinity of the site is approximately 1-foot above the tide level of Puget Sound with tidally related fluctuations in groundwater levels varying as much as 11 feet.

3.2 SEISMIC CONSIDERATIONS

The seismicity of the Puget Sound Region is attributed to the interaction between the Pacific, Juan de Fuca, and North American Plates. The following three significant sources of earthquakes contribute to the hazard at the site:

- Interplate subduction zone events near the coastline (last event approximately 300 years ago).
- Deep intraslab subduction zone events (1949, 1965, and 2001 events).
- Shallow crustal events near the site (last major event on Seattle Fault approximately 1,100 years ago).

For this technical memorandum, the following three levels of seismic events will be used:

- Minor seismic events – Return periods of 95 years (10 percent chance of exceedance in 10 years).
- Moderate seismic events – Return periods of 190 years (10 percent chance of exceedance in 20 years).

- Major seismic events – Return periods of 475 years (10 percent chance of exceedance in 50 years).

3.2.1 Liquefaction

Report Nos. 1, 4, 11, and 14 have assessed the liquefaction potential at the site. The amount or severity of liquefaction is related to the level of acceleration and earthquake magnitude used in the analysis. Report Nos. 1 and 11 performed a liquefaction analysis based on acceleration levels of 0.1 to 0.2 g, which correspond to an event smaller than the 475-year design event. These studies found the depth of liquefaction (i.e., the depth to which liquefaction will likely occur) to be about 15 to 20 feet. Report Nos. 4 and 14 performed a liquefaction analysis on the 475-year design event and found that the depth of liquefaction to be about 20 to 40 feet.

Report No. 14 also estimated the amount of liquefaction-induced settlement from the 475-year design event. The estimated settlement ranged from 1 to 12 inches with the highest settlements calculated in the waterway.

3.2.2 Lateral Spread

The occurrence of liquefaction can lead to lateral spreading along the banks of the river. Report No. 11 estimated that lateral ground displacement caused by liquefaction induced lateral spreading could be on the order of 6 inches for the 95- to 190-year design events based on Youd et al. (1999). Report No. 14 estimated the displacement from lateral spreading on all eight of their new soil borings with a range of 3 to 30 feet for the 475-year design event based on Youd et al. (2002).

3.2.3 AASHTO Soil Profile Type

There seems to be a disagreement in the literature regarding the AASHTO soil profile type. Report No. 11 does not specifically give a recommendation for the AASHTO soil profile type; however, they include Type II in a figure of response spectra. Report No. 14 states that an improved soil block around foundation elements will improve the AASHTO soil profile to Type IV or possibly Type III. For preliminary design, Report No. 14 recommended Type IV for foundation elements within the improved soil zones.

3.2.4 February 28, 2001, Nisqually Earthquake

Report Nos. 11 and 14 discuss the results of the February 28, 2001, Nisqually Earthquake on the site. The Nisqually Earthquake was a magnitude 6.8 event that occurred at a depth of 33 miles in the vicinity of Olympia, Washington. Accelerations were recorded throughout the greater Puget Sound vicinity. Peak horizontal ground accelerations of 0.17 and 0.25 g were recorded within 1-mile of the site at Boeing Field. After the earthquake, evidence of liquefaction was observed at the south approach of the bridge. Report No. 11 indicates that minor movement (i.e., less than 2 inches) occurred near the site and that the south approach may have shifted to the north by about 1-inch, as concluded through observation of cracks in nearby paved areas and difficulties with closure of the bascules. In addition, Report No. 11 states that localized settlement of the roadway may be on the order of 1 to 2 inches. Report No. 14 mentions that the lateral spreading could have induced undesirable lateral loads on the existing pile foundations and that two bents along the south approach had to be shored up with new timber bents and the south abutment fill had to be grouted to fill voids that had developed under the south approach roadway slabs.

3.3 EXISTING PILE CAPACITY ISSUES AT BASCULE PIERS

3.3.1 Design Capacity and Loads

According to Report No. 1, the working design load for each pile was assumed to be from 22 to 30 tons. The design load was called out as 22 tons in Report No. 11.

3.3.2 South Bascule

The south bascule is supported on piles that were driven to elevation -73 feet (site datum), which is into the glacially consolidated soils. The south bascule piles were typically driven to practical refusal of less than 1/8-inch per blow per inch with just 5 percent driven to a final resistance of 1/8- and 1/4-inch per blow. According to Report No. 1 these piles would have an ultimate compressive capacity of 50 tons. Using the ultimate pile capacity figure from Report No. 11 the piles have 90 to 110 tons ultimate compressive capacity for 1/4- and 1/8-inch per blow, respectively. In addition, Report No. 11 states that a capacity of 25 tons may be added to the capacities from the figure due to pile pore water dissipation after pile driving. The literature indicates that the south bascule piles have adequate soil capacity.

3.3.3 North Bascule

The north bascule is supported on piles that were jetted to approximately elevation -84 feet (site datum) and terminated at approximately elevation -97 feet. At elevation -97 feet, the piles are terminated in soils described as very stiff clayey silt (Report No. 1) or estuarine sand deposit or weather glacial till (Report No. 11). The north bascule piles were typically driven to 0.5 to 2 inches per blow as shown in a figure in Report No. 1. Ten of the 320 piles (3 percent) showed greater than 2 inches per blow.

Report No. 1 used the Engineering News Formula to estimate pile capacities of 6 to 12 tons during initial driving with an increase to 12 to 24 tons about 1-year following construction. After initial settlement of the pile, they state the capacity could reach 32 to 54 tons.

Report No. 11 estimated that 97 percent of the piles have a capacity of 40 tons or greater based on the pile driving formula developed by the Washington State Department of Transportation (WSDOT). According to Report No. 11, only one of the 315 piles had a penetration resistance that would indicate a factor of safety of one or less based on the design load of 22 tons. These capacities neglect the additional 25 tons of capacity from pore water dissipation that Report No. 11 estimates. Report No. 11 states that the piles supporting the north bascule may be assumed to have an ultimate compressive capacity of 65 tons (40 tons from the driving formula and 25 tons from pore pressure dissipation).

3.4 NEW ANALYSES

3.4.1 AASHTO Soil Profile Type

AASHTO calls out Soil Profile Type I as characterized by a shear wave velocity greater than 2,500 feet per second and Type IV as less than 500 feet per second. Report No. 14 performed shear wave velocity tests in Borings SB-2 and SB-6 (i.e., one on each side of the bridge). We calculated that the average shear wave velocity within the upper 100 feet of Borings SB-2 and SB-6 as 856 and 818 feet per second, respectively. In our opinion, these shear wave velocities are consistent with AASHTO Soil Profile Type III or 2003 International Building Code (IBC) Site Class D. The selected AASHTO soil profile type has an impact on how much the site soils amplify the ground accelerations, as shown in Table 3.1 in Section 3.4.4. This affects the magnitude of the loads expected from the seismic event.

3.4.2 Lateral Spread

The 6 inches of lateral spread under the 95- to 190-year events calculated by Report No. 11 seems reasonable. The 3 to 30-foot of lateral spread calculated by Report No. 14 also can be expected from the equations for the 475-year design event. These equations are not very accurate and tend to calculate very conservative (i.e., high) displacements at conditions very near or less than a slope stability factor of safety of 1.0.

3.4.3 Existing Pile Capacity

Hart Crowser performed an independent analysis of the pile capacity using the Danish Driving Formula. The following assumptions were used in the analysis:

- Vulcan 1 Hammer with 5 kip striking weight and operating at 60 blows per minute.
- Hammer operating at 80 percent efficiency.
- 70-foot-long Douglas Fir piles with 7-inch tip and 14-inch head.

Based on a final resistance of 2 inches per blow (97 percent of the piles had this or better), using the above assumptions, we calculated the pile capacity to be 35 tons based on initial driving. Using the skin friction estimate of 20 to 30 tons of prior studies (Report Nos. 1 and 11), we estimate an ultimate pile capacity of 55 to 65 tons. This is at or slightly below the 65-ton estimate from Report No. 11, but above the interpreted 32- to 54-ton capacity from Report No. 1. We agree with conclusions drawn by Report No. 11 that the foundations for the bascules have performed satisfactorily to date and can adequately support the existing static loads of 22 to 30 tons.

Some of the reports questioned the pile capacities based on the method of installation. The piles were jetted to within 14 feet of the design tip elevation. The jetting may temporarily reduce the side friction along the pile during driving, but should not have any significant long-term effects on the pile capacity.

3.4.4 Site-Specific Accelerations

An analysis was performed to estimate the site-specific ground surface acceleration for a number of different average return periods. The probabilistic seismic hazard analysis from the 2002 National Seismic Hazard Mapping Project (USGS 2002) was used to determine the site-specific seismic hazard for a rock site (IBC Site Class B or AASHTO Soil Profile Type I). The USGS was extrapolated to return periods below the last available data point at 108 years using straight-line extrapolation on a log-log plot. Based on the trend of the data, this is slightly conservative (i.e., it produces slightly larger accelerations than one would expect), but is a good estimate.

The actual site conditions are more consistent with AASHTO Type III. The AASHTO Type III is generally consistent with the Site Class D of the IBC. We used IBC relationships to convert the USGS rock site accelerations to IBC Site Class D soil accelerations at the ground surface. This is a simple and reasonable means of obtaining ground soil accelerations at the site. The selected AASHTO soil profile type has an impact on how much the site soils amplify the ground accelerations. This in turn affects the magnitude of the loads expected from the seismic event.

Site-Specific Peak Ground Accelerations		
Return Period in Years	IBC Site Class B (AASHTO Type I)	IBC Site Class D (AASHTO Type III)*
10	0.04 g	0.07 g
20	0.06 g	0.10 g
50	0.10 g	0.17 g
95	0.15 g	0.23 g
190	0.21 g	0.30 g
475	0.34 g	0.41 g

Table 3.1 - Ground Surface Accelerations for Various Average Return Periods

*South Park Bridge soil type.

3.5 CONCLUSIONS

Report Nos. 1, 4, 11, and 14 assessed the soil conditions at the South Park Bridge site. There are two conflicting observations coming from these reports. The areas of disagreement occur in the assumed pile capacities of the bascule piers and the identification of the soil profile.

From the results of the new analysis conducted for this report, the conclusions of Report No. 11 most accurately predict the pile capacities. Thus, it can be assumed that the bascule pier piles have performed satisfactorily to date and can adequately support the existing static loads on the structure.

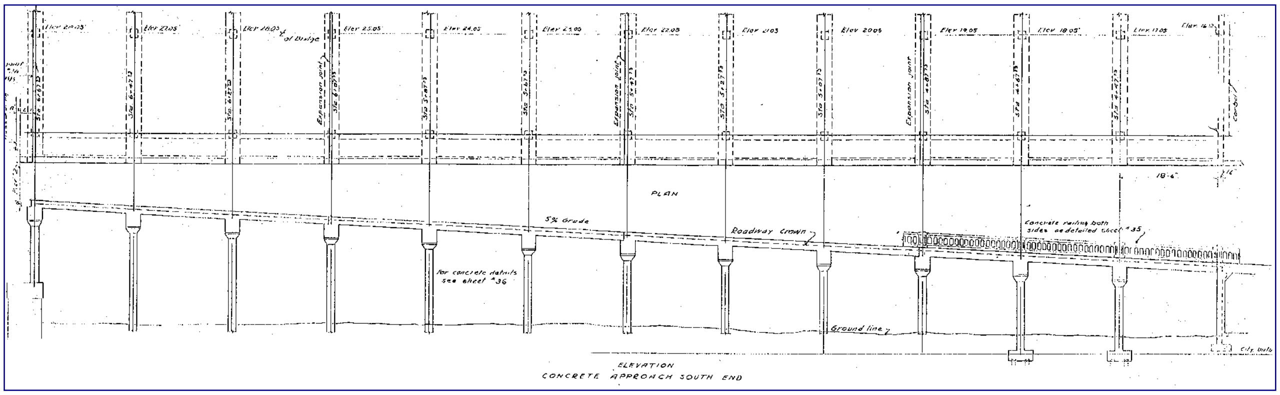
The selection of the appropriate soil profile is important because it defines how much the site soils amplify the ground accelerations. This in turn affects the magnitude of the loads expected from a given seismic event. Report No. 11 references soil profile Type II in the calculations. Analysis performed for this report show that the actual site conditions are more consistent with Type III. This means that the actual site conditions will exhibit higher accelerations and larger loads than those assumed in Report No. 11. Consequently, these assumptions should be revisited prior to the selection of any retrofitting option.

4. CONCRETE APPROACH SPANS

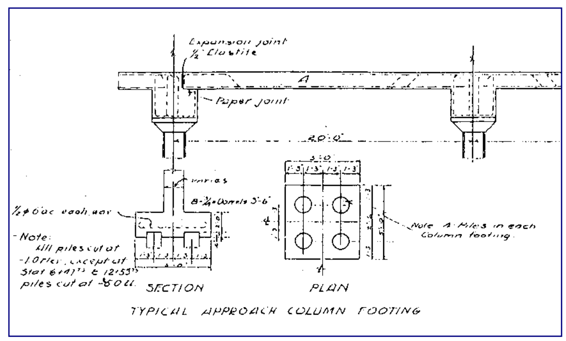
4. CONCRETE APPROACH SPANS

4.1 DESCRIPTION

The South Park Bridge concrete approach spans are reinforced concrete one-way slabs supported by concrete columns that are founded on timber piles. Both the south and north approaches consist of 12 spans approximately 20 feet in length for a total length of approximately 240 feet each.



4.2 SUBSTRUCTURE



4.2.1 Description

Each pier consists of a concrete pier cap supported by three concrete columns. A pile cap founded on four timber piles supports each column.

4.2.2 Previous Study Observations/Condition Assessment

(Relevant Reports: 4, 6, 11, 12, 14, and 15)

4.2.2.1 Seismic Study

Report Nos. 4, 11, 14, and 15 examined the soil conditions in the vicinity of the concrete approaches. Each of the reports indicates that the soils near the approaches are both liquefiable and subject to lateral spreading during an earthquake.

Report No. 11 studied the effects of an earthquake on the existing structure by modeling both the soil and structure. The report concludes that the columns of the concrete approach spans will be severely damaged in a minor seismic event, may collapse during a moderate seismic event, and will collapse during a major seismic event. This is an important issue, because if the bridge were designed to current standards, it should not only survive the moderate event without collapse but should not collapse under a major event.

The Nisqually earthquake that occurred on February 28, 2001, had a magnitude of 6.8 with ground motions equivalent to a moderate seismic event. The resulting damage to the south concrete approach structure included ground settlement and cracking in the columns. The bridge deck had to be shored with timber frames because the structural capability of the cracked substructure and superstructure was uncertain. Report Nos. 11 and 12 document cracks that developed around the perimeter of many columns, suggesting the beginning of plastic hinging. Bridges designed to current standards design the plastic hinge zones to prevent failure mechanisms that would lead to collapse; however, bridges of the 1930s typically would not have this level of protection and would be susceptible to this failure mechanism. If the Nisqually earthquake lasted longer, much more damage would have happened to the structure.

4.2.2.2 Structural Observations

4.2.2.2.1 Timber Piles Rehabilitation

Deterioration was found in the top portion of some of the timber piles for the concrete approach structures. Some concrete approach structure foundation piles were rehabilitated in the 1970s. The rehabilitation consisted of cutting off deteriorated pile tops and encasing the pile tops in a new concrete cap.

4.2.2.2.2 Concrete Condition

Several investigations were made into the condition of the approach structure concrete. Tests were conducted for strength and possible forms of chemical degradation.

4.2.2.2.2.1 Strength

The strength of concrete is determined from compression tests on standard cylindrical core specimens. Tests were conducted on six cores taken from the concrete approach pier caps. The overall average compressive strength was 4,840 psi. This compressive strength is likely adequate for the original design, but existing pier reinforcement does not meet current seismic design criteria.

4.2.2.2.2.2 Petrographic Analysis

Petrographic analysis can be used to determine chemical and physical irregularities in concrete. This type of information can aid in the determination of the root cause of failure and extent of cracking, scaling, or delamination.

A representative core was prepared and tested in accordance with ASTM C856, *Petrographic Examination of Hardened Concrete*. The petrographic analysis results showed evidence of the effects of alkali silica reactions (ASR). This is a condition where silica present in certain aggregates, either rocks or sand, combines with water to form a gel that absorbs water and expands, potentially damaging the concrete.

4.2.2.2.2.3 Chloride Ion Concentration

For reinforced concrete bridges, one of the major forms of environmental attack is chloride ingress, which leads to corrosion of the reinforcing steel and subsequent reduction in the strength, serviceability, and aesthetics of the structure. Test results of the approaches showed levels of water-soluble chloride ions ranging from 306 to 610 parts per million. It was noted in the reports that FHWA guidelines consider levels higher than 500 parts per million sufficient reason to replace a deck slab. Although the concrete may be able to continue supporting the structure, the levels of deterioration are such that it is unreasonable to continue repairing the concrete and it is time to plan for replacement.

4.2.2.2.2.4 Spalling and Cracking

Cracking and spalling are exhibited throughout the substructure. This can be caused by corrosion of reinforcing steel, freeze-thaw, or aggressive chemical attack. Additionally, the exposed reinforcement has undergone moderate corrosion.

During concrete repair work in 1995, the contractors, in an attempt to chip away deteriorated concrete down to reliable concrete, chipped away almost an entire pier cap cantilever. It was discovered that the concrete deterioration was from uniformly poor material throughout the concrete section. This supports the test results that identified the concrete being under attack by many deteriorating mechanisms, especially ASR.

4.2.3 Review Comments and Recommendations

The recommendation stated in Option II of Report No. 12 to replace the concrete approach structures in their entirety is a valid recommendation for the following reasons:

- Seismic vulnerability – The existing structure was not designed to service even a moderate earthquake with the Nisqually Earthquake exposing the vulnerability of the structure. Currently at least two spans of the south concrete approach structure can no longer reliably support themselves and columns are showing signs of plastic hinging.
- Concrete in state of progressive decay – Report No. 16 observes that a number of deterioration mechanisms are acting simultaneously throughout the substructure. As noted in Report No. 6, the American Concrete Institute Report, ACI 201.2R, *Guide to Durable Concrete*, states the following in reference to the types of chemical attack that are currently effecting the substructure condition:

  > *"There are no known methods of adequately preserving existing concrete which contains the elements that contribute to the previously described chemical reactions."*

  Given that no methods of adequate repair are known, continued deterioration would be expected to develop even after attempting repairs. As a result, further rehabilitation is unreasonable.

It is recommended that plans for replacement or closure of the bridge begin immediately. As part of this process, it is recommended that work be done to provide a minimum level of protection from collapse. Full seismic retrofitting is not economically reasonable, but structural strengthening to prevent collapse during at least a moderate seismic event is recommended. This may include shoring of the footings and cross bracing of the columns similar to recommendations made in Report No. 11.

4.3 SUPERSTRUCTURE

4.3.1 Description

The superstructure is divided into three span-continuous segments. Each segment has three 20-foot spans for a total of 60 feet. An expansion joint separates each three span segment.

4.3.2 Previous Study Observations/Condition Assessment

(Relevant Reports: 6, 7, 11, and 12)

The concrete deck exhibits the cracking, spalling, and reinforcement corrosion that are typical of the substructure. Further exacerbating this condition is the fact that Boeing at one time stored chemical tanks under the bridge spans that are believed to have created an atmosphere that added to the concrete degradation.

Concrete spalling and exposed reinforcing steel on underside of deck

4.3.2.1 Load Rating

Load rating calculations are an analytical means to determine the capacity of a structure. If the resulting load rating value is above 1.0, the structural component being analyzed is assumed capable of supporting the load applied in the calculations.

The load rating analysis uses six truckloads to rate a bridge. There are three legal truckloads used to determine posting limits, AASHTO 1, 2, and 3. An HS20 vehicle is a hypothetical vehicle based on actual trucks and used for design, but is not an existing truck. The two overload vehicles represent extremes in the limits of permitted vehicles in Washington State.

The concrete approach spans were load rated in 1994. The load rating bounded the structural conditions by analyzing the structure utilizing the full depth as-built section (12-inch slab) and a partial depth section (9-inch slab) representing witnessed delamination and section loss in the deck slab. In general, the AASHTO 1, 2, and 3 trucks rated at greater than 1.0, but are marginally passing. The HS20, Overload 1, and Overload 2 trucks all rated below 1.0 for the slab and just over 1.0 at 1.1 for the crossbeam.

	12-inch Slab	9-inch Slab	Crossbeam
HS20	0.90	0.78	0.80
AASHTO 1	1.03	0.97	1.10
AASHTO 2	1.19	1.06	1.50
AASHTO 3	1.27	1.24	1.70
Overload 1	0.97	0.98	1.10
Overload 2	0.86	0.79	1.10

4.3.3 Review Comments and Recommendations

Like the substructures, the recommendation stated in Option II of Report No. 12 to replace the concrete approach structures in their entirety is again a valid recommendation. In the interim, it recommended that immediate action be taken to determine the capacity of the existing superstructure.

Since the 1994 load rating analysis, 11 more years of deterioration have taken place. After visiting the site as part of this peer review, the concrete section used for the previous load rating analysis appears to no longer represent the existing condition of the bridge. The concrete structure has continued to deteriorate and this progressive deterioration brings into question the actual ability of the reinforced concrete deck slab to perform. The previous load rating results were marginal. Any additional reduction in the assumptions previously made would suggest that the structure might no longer be load rated for current traffic.

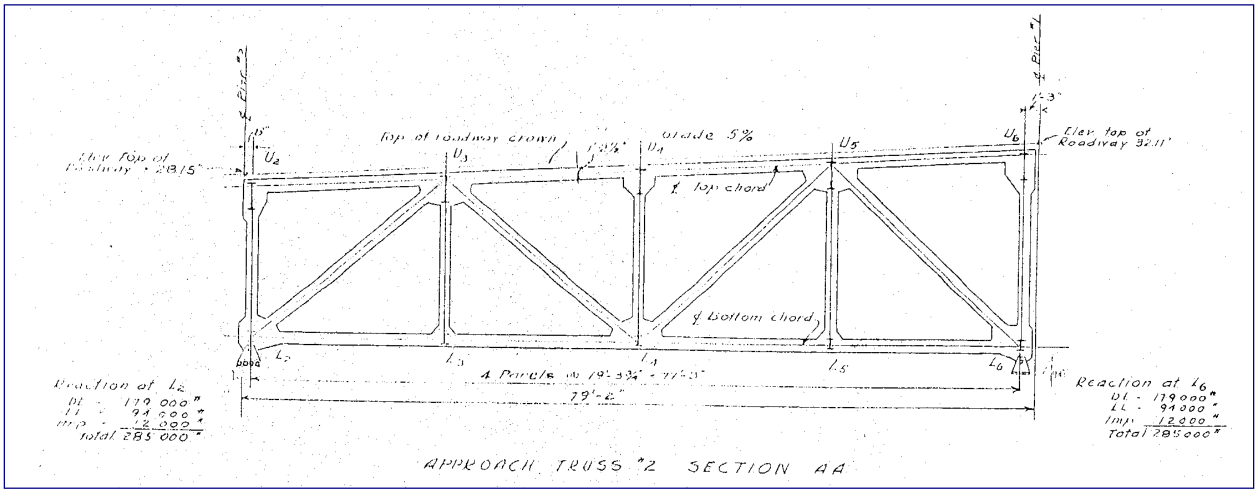
The only load rating method that eliminates the uncertainty related to the current deck deterioration would be a physical load test. For a physical load test, the bridge is loaded with an actual physical load and the structure's performance is measured. Typical reasons for recommending a physical load test are damage to structural elements, questionable or insufficient information on as-built plans, and/or any physical characteristics that affect performance but cannot be adequately accounted for in an analytical evaluation. All of these factors apply to the South Park Bridge, so it is recommended that a physical load test be performed to assist in verifying the assumptions made in previous analytical ratings and establish a more accurate load capacity. Given the progressive deterioration of the structure, it is recommended that load testing be done on a regular basis. The information from these load tests will likely result in posting the bridge for reduced loads.

5. STEEL TRUSS SPANS

5. STEEL TRUSS SPANS

5.1 Description

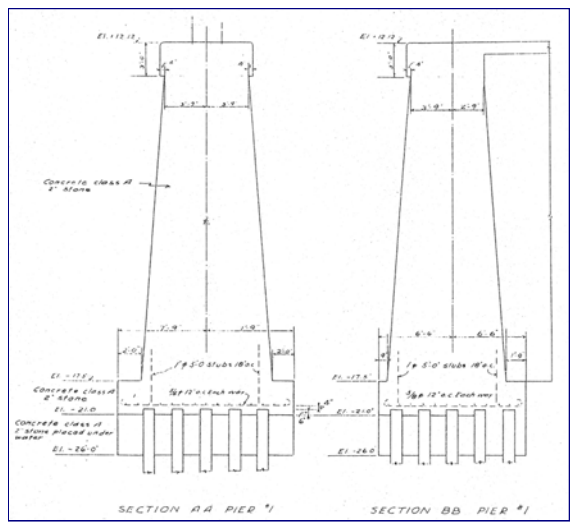
On either side of the bascule span, a two span steel truss structure bridges between the concrete approaches and the bascule piers. The individual spans are approximately 79 feet and 88 feet for a total of 167 feet.



5.2 Substructure

5.2.1 Description

Two concrete piers and one bascule pier support the steel trusses. The concrete piers are supported on timber piles. One pier consists of two unreinforced columns connected by a cap and a spandrel beam. The other pier has three unreinforced supporting columns connected by a cap and reinforced wall.



5.2.2 Observations/Condition Assessment

(Relevant Reports 3, 8, 9, 10, 11, and 12)

The soil conditions in the vicinity of the steel truss approach foundations are the same that exist throughout the project site. There is a potential for liquefiable soils in the upper 40-foot layer. This condition could lead to lateral spreading of up to 30 feet and vertical settlements up to 2 feet during a major seismic event.

Condition of the concrete piers has been rated as fair. Hairline and map cracking is prevalent on all exposed faces. Deterioration and softening of concrete has been encountered on the top of footings and several corners have spalled. Repairs have had mixed success, with some repairs intact and others delaminating.

Demand-to-capacity ratios for specific components of the substructure are calculated in Report No. 11 and listed in the chart below. In this analytical calculation, values exceeding 1.0 show that the structural component being analyzed may not be able to support the given load, while values below 1.0 imply the structural component being analyzed would be able to support the given load.

	Pier 1		Pier 2	
	95-Year Event	190-Year Event	95-Year Event	190-Year Event
Pile Foundations	1.30	1.96	0.80	1.60
Columns		0.65		
Cap Beams	2.3	2.7		

Report No. 11 concludes that the footings may fail resulting in possible collapse of the pier columns due to overturning during a moderate seismic event and will collapse during a major seismic event with the cap beams likely to fail during even a moderate earthquake. Though the columns have a demand-to-capacity ratio below 1.0, the report anticipates that significant damage will likely occur during a moderate seismic event. This is an important issue, because if the bridge were designed to current standards, it should not only survive a moderate seismic event without collapse but should not collapse under a major seismic event.

5.2.3 Review Comments and Recommendations

The recommendation stated in Option II of Report No. 12 to replace the concrete piers supporting the steel deck trusses in their entirety is a valid recommendation for the following reasons:

- Seismic vulnerability – The existing structure was not designed to service even a moderate earthquake. Consequently, there is no reinforcing steel in the pier columns.
- Concrete in state of progressive decay – As in the concrete approach substructure, a number of deterioration mechanisms are acting simultaneously throughout the concrete of the steel truss substructure. Though thorough testing of the concrete has not taken place at the steel truss approach structures, it is likely that there are no methods of adequate repair and continued deterioration would be expected to develop even after repairs. As a result further rehabilitation is unreasonable.

It is recommended that plans for replacement or closure of the bridge begin immediately. As part of this process, it is recommended that work be done to provide a minimum level of protection from collapse. Full seismic retrofitting is not economically reasonable, but structural strengthening to prevent collapse during at least a moderate seismic event is recommended.

5.3 SUPERSTRUCTURE

5.3.1 Description

The superstructures are steel trusses supporting a concrete deck on the top of the truss framing. Each truss has a fixed bearing at one support and a rocker bearing at the other. The rocker bearing is inoperable due to corrosion. Lateral bracing exists only at the fixed supports. No lateral restraint exists at the rocker bearings.

5.3.2 Observations/Condition Assessment

(Relevant Reports: 3, 8, 9, 10, 11, and 12)

All primary structural elements are considered sound. The common concern throughout the reports is the inadequate restraint and the absence of a complete lateral load path to transfer seismic loads from the deck level to the supports. Report No. 11 concludes that the vertical truss members will be severely damaged in a minor seismic event, may collapse during a moderate seismic event, and will collapse during a major seismic event. Had the Nisqually earthquake lasted longer, this may have occurred. This is an important issue, because if the bridge were designed to current standards, it should not only survive the moderate seismic event without collapse but should not collapse under a major seismic event.

5.3.3 Steel Truss Approach Spans

The 1996 bridge load rating from Report No. 8 is summarized below:

	Approach Steel Truss	
	Span 1	Span 2
HS20	1.89	1.13
AASHTO 1	2.52	1.52
AASHTO 2	2.32	1.53
AASHTO 3	2.40	1.66
Overload 1	2.54	1.54
Overload 2	1.94	1.38

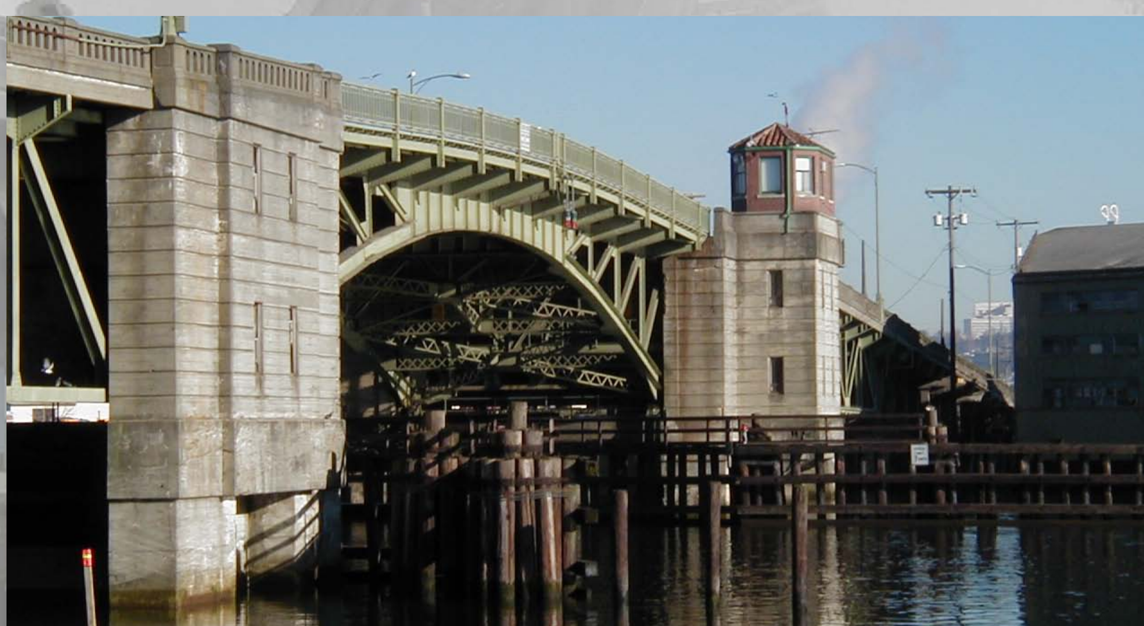
Values greater than 1.0 indicate that the steel trusses are adequate to carry the vehicle loads currently used for the evaluation of bridge structures.

5.3.4 Review Comments and Recommendations

Like the concrete approach spans, the steel trusses are continuing to deteriorate. The trusses should be replaced with the replacement of the remainder of the bridge.

In the interim, the trusses should be load tested along with the concrete approaches. It is also recommended that work be done to provide a minimum level of protection from collapse. Full seismic retrofitting is not economically reasonable, but structural strengthening to prevent collapse during at least a moderate seismic event is recommended. This would include strengthening vertical truss members that are susceptible to collapse during a moderate earthquake.

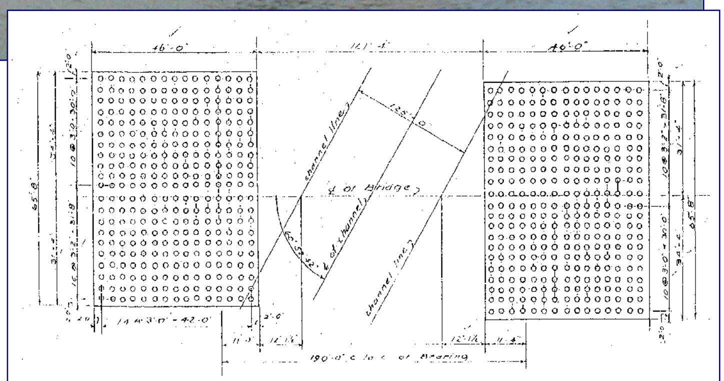
6. BASCULE SPAN



6. BASCULE SPAN

6.1 Description

The bascule span is a Scherzer Rolling Lift double leaf movable span. The double leaf bascule has a center-to-center distance between the front bearings of 190 feet. The length of roll for each leaf is 18 feet. The bascule leaves are riveted steel trusses with center-to-center spacing of trusses equaling 36 feet. The bascules are supported on reinforced concrete piers founded on timber piles. The bascule spans are opened an average of three to five times per day.



REVISED BASCULE PIERS PILE PLAN

6.2 Substructure

6.2.1 Description

The bascule pier foundations are approximately 66 feet by 46 feet with a 6-foot thick pile cap above a 15-foot thick seal. The foundations are supported by a group of 315 timber piles.

6.2.2 Observations/Condition Assessment

(Relevant Reports: 1, 2, 5, 9, 10, 11, 12, and 15)

The major concern regarding the bascule pier substructures is that the timber piles were driven into an alluvial deposit layer generally consisting of very soft soils ranging from approximately 30 to 60 feet thick. These soil conditions possibly have led to the binding of the bascule leaves through reduced pile capacity leading to differential settlement between the bascule piers, differential settlement within the north bascule pier foundation causing cracking, and/or rotation of the piers during operation of the bascule leaves.

Driving records show that the north bascule pier piles were not driven to refusal in the glacial till unlike the south bascule pier piles, which were driven to refusal. No reasons were given to explain why the timber piles in the north bascule pier were stopped in a relatively soft layer. This condition has led to speculation that the north bascule pier is moving more than the south bascule pier.

Assumed pile capacities varied widely from report to report. In Report No. 2, the pile capacities assumptions actually varied within the report. Report No. 1 estimates the capacity of the north bascule pier piles as low as 6 to 12 tons. This is well below the probable maximum design loads of 22 to 30 tons calculated in Report No. 2. Report No. 1 goes on to provide an analysis that results in possible allowable pile capacities for the south bascule piles of 50 tons and the north bascule piles having ultimate capacities as high as 30 to 50 tons. Report No. 11 calculated ultimate pile capacities of 65 tons.

Report No. 2 conducted new soil explorations. The new soil boring data was used in a model of the north bascule pier foundation. The model analysis predicted that significant settlement of the north bascule pier likely occurred during, and shortly after, construction. Using the same data, an analysis was performed to determine the impact of liquefaction of the soft layer on the foundation. The analysis predicted that loss of load capacity particularly for the outer piles might occur.

While Report No. 1 is often referenced in other reports for its estimated low pile capacities, the report goes on to explain that a low factor of safety is not the most critical factor for a foundation like the bascule piers with uniform loading on a large pile group supported by a thick reinforced concrete cap. If the loads exceed the ultimate pile group capacity, the pier would settle an additional amount until equilibrium is achieved. However, it states if such a pile group is subject to eccentric loads, as is the case with the bascule leaves, differential settlement could cause tilting of the foundation.

Report No. 1 modeled the 315 individual piles to determine the possible settlement of the bascule piers. The model simulated a situation in which the outer piles lost part of their capacity either from liquefaction or yielding due to high loads. Settlement contours are created for both a pier cap that had cracked and no longer acted as a unit and one that remained intact as designed. The differential settlements resulting from the model ranged from 0.77 to 1.03 inches for the uncracked pier and 1.10 to 1.93 inches for the cracked pier.

Interestingly, Report Nos. 1 and 11 interpreted tiltmeter data as showing the foundations performing satisfactorily.

6.2.3 Review Comments and Recommendations

There are many reports discussing the movement of the bascule piers, however, no conclusive evidence is presented to define the root cause of the pier movement. The conclusions for movement vary widely in the foundation studies. Possible reasons given for the movement of the piers are as follows:

- Piles not driven to refusal and hung up in a soft layer creating differential settlement.
- Deteriorating and/or failing timber piles.
- Poor concrete condition and associated cracking in the superstructure rather than in the foundations.

The fact that the north bascule pier piles were not driven to refusal, but were left hanging in a soft layer is a reason given for the settlement and poorer condition of the north versus the south bascule pier. However, the tiltmeter information and various surveys of the structures show that both piers are exhibiting movement within the same order of magnitude. This would suggest that a condition common to both piers might be creating the movement.

Report Nos. 2, 12, and 15 all state that the north bascule pier movement is related to the premise that the piles have a capacity of 6 to 12 tons as reported in Report No. 1. However, Report No. 1 also states that the ultimate pile capacity of the north bascule timber piles is actually between 55 and 65 tons. This brings the north bascule timber pile capacity in line with the assumed south bascule timber pile capacities effectively nullifying the stated differences between the south and north bascule pier foundations.

Deterioration and/or failure of the timber piles were not physically verified in any of the reports. The fact that the timber piles are completely embedded in concrete and are below the mud line would imply that the common mechanisms that cause deterioration such as insects or mold are not likely to occur in this environment due to the lack of oxygen. However, Report No. 1 mentions that dredging of the waterway channel could have reduced the pile capacity and exposed the exterior piles to marine organisms, but there is no verification of this occurring in the numerous underwater inspection reports (Report Nos. 9, 17, 18, and 20).

The movement does not appear to be affecting the structural performance of the bascule span. Due to the shear size of the foundations, the bascule foundations are not expected to collapse even during a major seismic event. However, the movement is having a significant impact on the operation of the bascule leaves. Clearances provided between the bascule leaves are not sufficient to accommodate the movement the piers are experiencing. This is discussed further in Section 6.4.

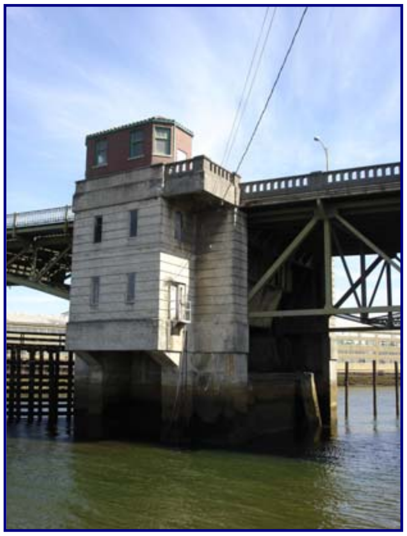
The recommendation to replace the bascule piers is valid for the following reasons:

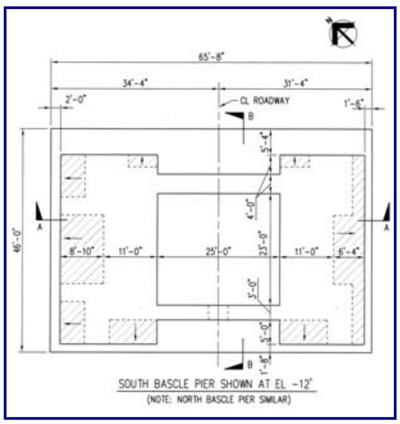
- Like the approaches, the bascule piers are experiencing continuing deterioration. The size of the piers has reduced the structural impact of that deterioration, but the mechanism of deterioration likely cannot be repaired.
- The inability to define the causes of the movement, the continued movement of the piers and the inadequate clearances at the bascule leaves are significantly impacting the operation of the bascule leaves.

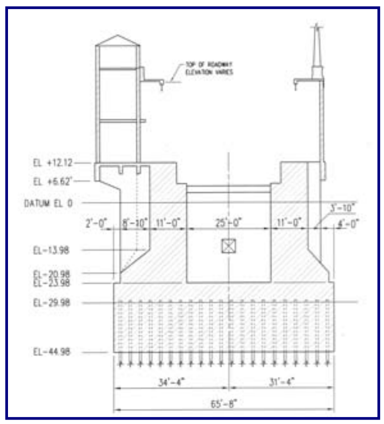
6.3 PIERS

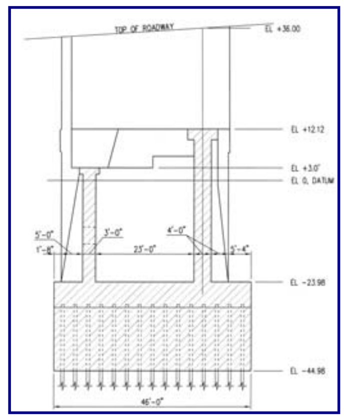
6.3.1 Description

The two bascule piers support the counterweights, operating machinery, and the operator's control room. The east and west bascule pier walls are 11 feet thick unreinforced concrete gravity structures. The north and south bascule walls are 3- to 4-foot thick lightly reinforced concrete wall structures.









6.3.2 Observations/Condition Assessment

(Relevant Reports: 2, 5, 9, 10, 11, 12, and 15)

The main concern for the bascule piers is the deteriorating condition of the surface concrete and the fact that both piers have experienced considerable cracking. The level of deterioration significantly increased between the 1986 to 1994 inspections with further deterioration witnessed in the 1997 inspection. Spalling on the exterior face of the piers ranges in size from 4 square feet to 100 square feet and have depths up to 1-foot. A clear demarcation exists at elevation –10 with concrete conditions fair above and poor below this elevation. In 1999, the Washington State Bridge Inventory System (WSBIS) Bridge Inspection Report rated the substructure a score of 5 out of a possible score of 8. In 2000, this score fell to 4, then to 3 in 2001. As of 2002, the score remained a 3. The 2002 report stated that the loss of section, deterioration, and spalling had seriously affected primary structural components. It went on to say that local failures are possible.

Report No. 2 reports on post tensioning that was done in the early 1980s to stop movement of the bascule walls. Apparently, the walls were moving in opposite directions transversely away from the centerline of the structure. The report states that movement of the walls was likely the cause of the track girders binding on the track supports at the North Bascule leaf. The walls were post tensioned to arrest this movement. Movement was stabilized until 1987 when binding began again. Assuming that the track supports were in proper alignment when constructed, the report states that settlement and associated cracking of the pier has contributed to the movement of the mechanical components.

Tiltmeters were installed in an attempt to track the pier movements. The tiltmeters measure rotational movement in the longitudinal (north-south) and transverse (east-west) directions. The first system was installed in September 1991, and monitored movement for six months. In June 2001, two more tiltmeters were added to each pier.

Representative concrete section loss on exterior of bascule piers

Report No. 11 analyzed the structure seismically. The severe vertical cracking in the north and south bascule walls was modeled by eliminating elements in the model to represent a vertical cracks. It was found that the existing cracks in the north and south walls reduced the connectivity of the four bascule walls causing the walls to no longer form a box to resist seismic forces. The result is that tensile forces increase and cracks are more likely to form during a seismic event. The report concludes that seismic events could be the reason for cracks in the east and west pier gravity walls.

Concrete cores were taken during several inspections. The cores were then tested for compressive strength and chemical analysis. The tests for compressive strength varied so widely that no apparent trend could be determined. Report No. 9 noted that the tests results had limited utility from an engineering perspective.

Report No. 19 determined that relatively high values of soluble chloride ions found in the concrete and the severe spalling of the exterior walls suggest the concrete failure was in part due to chloride induced corrosion of the reinforcement.

Report No. 9 referenced the large spalled areas underwater, with soft chalky crumbling concrete on the exterior surfaces, particularly at the corners. This type of deterioration was reported to be indicative of classic chemical deterioration of the concrete matrix, most likely sulfate attack, but also possibly attributed to alkali-silica reaction (ASR) or delayed ettringite formation (DEF). Further evidence refuting that the spalling was due to chloride induced corrosion was the fact that spalling goes well beyond the depth of the reinforcement. Petrographic analysis was suggested as a way to determine the root causes of the deterioration.

Spall repairs have had mixed success. In several areas, the bond between the repair concrete and the original concrete has deteriorated and the repairs have delaminated.

6.3.3 Review Comments and Recommendations

The recommendation to replace the bascule piers as part of the total bridge replacement is valid for the following reasons:

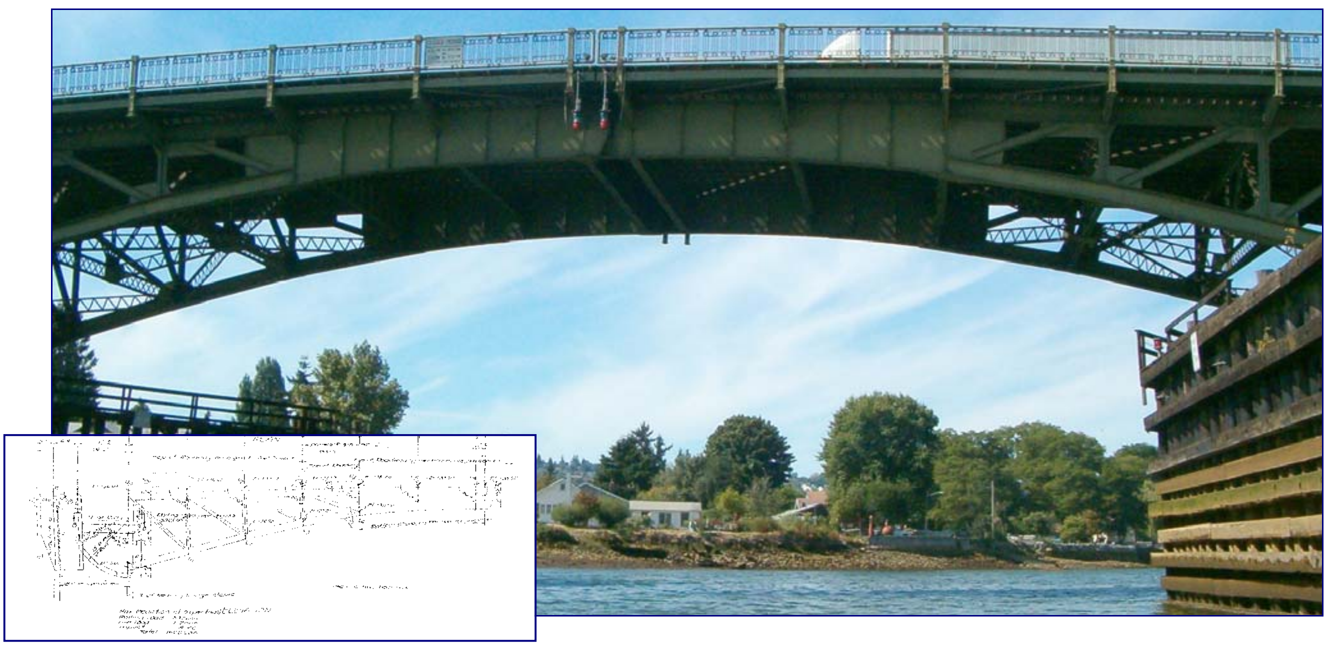
- Like the approaches, the bascule piers are experiencing continuing deterioration. The size of the piers has reduced the structural impact of that deterioration, but the mechanism of deterioration likely cannot be repaired.
- The walls of the operator house are vulnerable to severe damage in a minor seismic event and may collapse during a moderate seismic event. Had the Nisqually earthquake lasted longer, this could have taken place.

In the interim before replacement, it is recommended that work be done to provide a minimum level of protection from collapse. Full seismic retrofitting is not economically reasonable, but structural strengthening to prevent collapse of the operator house walls during at least a moderate seismic event is recommended. This would include bracing the operator house walls that are susceptible to collapse during a moderate earthquake. See Report No. 11 for possible retrofit schemes.

6.4 Bascule Leaves

6.4.1 Description

The bascule leaves form the movable portion of the bridge. This includes the steel truss superstructure, the steel rolling girders, and the counterweights. The existing bridge deck is an open-grated steel deck system.



6.4.2 Observations/Condition Assessment

(Relevant Reports: 2, 5, 9, 10, 11, 12, and 15)

The bascule leaves are the critical location where the pier and leaf movement impact the operation of the bridge. It is at this location where various movements will cause the bascule leaf center teeth and interlocking jaw locks at the tips of the moveable spans to conflict with one another during opening and closing. Misalignment can render the bascules inoperable. When this happens repairs must be made to allow the bridge to be opened and closed to accommodate both vehicle and navigable traffic.

Report No. 2 reports that the North Bascule track supports and racks are significantly out of alignment. The report concludes that if the misalignment is not corrected the bridge will likely become inoperable. The County has performed numerous adjustments to the bascule system, but no adjustment has been found that eliminates the ongoing alignment issue. Adjustments include shimming the leaves, trimming track lugs, installing strike plates at the center locks, etc.

The bascule superstructure was load rated for various leaf components. Results are detailed in the table to the right.

	Bascule	
	Truss	Plate Girder at Centerline Span
HS20	1.08	1.26
AASHTO 1	1.45	1.74
AASHTO 2	1.46	1.48
AASHTO 3	1.58	1.39
Overload 1	1.45	1.76
Overload 2	1.26	1.12

Values greater than 1.0 indicate that the component has the capacity to carry current traffic loads. However, continuing deterioration may reduce these ratings.

Of greater concern is the fact the bascule truss superstructure lacks sufficient lateral restraint at the interface with the concrete pier. Lateral loads are currently resisted by friction, bearing of the rolling truss on the guide tracks, or by the lugs on the guide tracks. In addition, the Scherzer rolling lifts have been known to wander on their steel bearing tracks after years of service and guide track wear.

6.4.3 Review Comments and Recommendations

No conclusive reasons were provided that explained the misalignment of the bascule components. Various reports reasoned that the cracked and/or settling structure was the cause, while other reports referred to the fact the Scherzer bascules have historically wandered on their steel tracks after years of service. It is not known if the misalignment is mechanical and/or structural in nature. It is recommended that an extensive survey of the piers be performed that ties the entire structure and the various components together. This will aid in determining the extent that the structure location and the mechanical configuration contribute to the operational problems and assist in determining the length of time until the bascule leaves are rendered inoperable.

During an earthquake, the counterweight has a tendency to move up, down, and transversely. Consequently, to prevent collapse during a moderate earthquake, the need for new lateral bracing for the counterweight and new shear blocks for the truss tracks should be researched.

6.5 Mechanical Systems

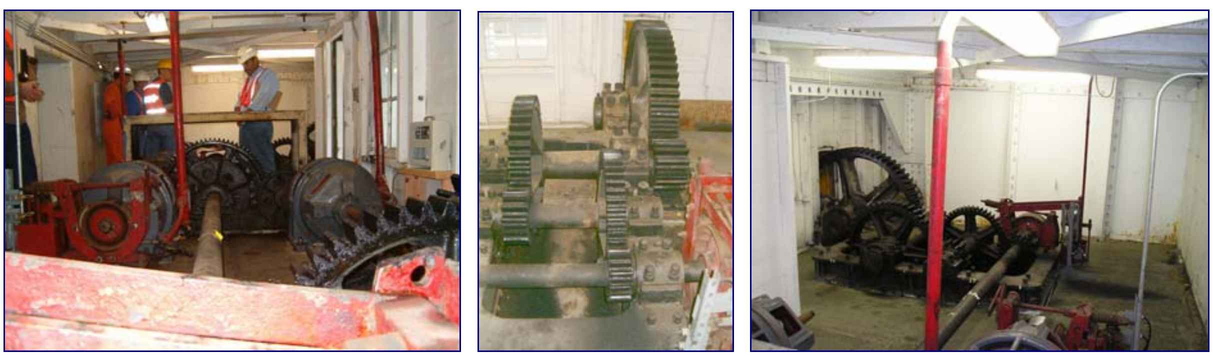
6.5.1 Description

Major mechanical components of this bridge include the span drive systems, the rolling racks, and the interlocking jaw locks. The span drive systems can be further broken down to motors, primary reducers, differentials, open reduction gearing, and drive rack and pinion.

Rolling end of bascule leaf and support track

Interlocking jaw locks

The pinion centerlines are collinear with the arc center of the rolling racks. In operation, the motor/reduction units cause the pinion to progress along the drive rack. A couple is formed with the reacting force of the rolling rack/track, causing the bridge to rotate as the pinion translates.



Bascule leaf room and machinery

6.5.2 Observations/Condition Assessment

The reports reviewed contained numerous condition studies of the bridge's mechanical systems. The scope of these studies varied from casual observation with a condition statement of opinion, to in-depth reviews using non-destructive testing (NDT) on key parts. In 1992, Stafford Engineering conducted a fairly comprehensive mechanical condition survey and found the machinery to be in generally good condition, but recommended immediate replacement of the pinion shaft bushings and repairs to the differentials. Both of these recommendations have since been accomplished and King County personnel report that the bridge machinery operates well.

6.5.3 Previous Report Recommendations

In addition to the corrective actions mentioned above, many recommendations have been made for improvements to the bridge mechanical systems. These range from relatively minor, such as safety covers and interlocks, to major additions, such as counterweight seismic stabilizers and rolling track restrainers that would be part of a comprehensive bridge renovation effort.

6.5.4 Review Comments and Recommendations

Recommendations for repair, upgrade, and renovation to bridge mechanical systems have been made that would improve operations and extend service life. None of the recommendations, however, significantly contributes to resolution of the more significant issues of foundation strength and seismic stability. These issues alone require complete replacement of the bridge.

6.6 ELECTRICAL SYSTEMS

6.6.1 Description

The power service entrance is located on the east side of the north approach. A 600A breaker with an adjustable solid-state trip serves as the main disconnect for all 480-volt power. A 120/240-volt single-phase service is provided for lighting and miscellaneous loads. The switchboard/control board is outdated and considered unsafe by today's standards. Light circuits and motor feeders between the two control houses are conducted in two submarine cables. The four main drive motors and two emergency drive motors are all 440-volt 3-phase, wound rotor motors. Direction of motor rotation is reversed by an arrangement of two pole contactors that switch phase legs to the motors, and speed control is accomplished by varying resistive loads in line with the motor windings.

6.6.2 Observations/Condition Assessment

The electrical system is largely original equipment. It has been maintained by scavenging parts made available when various control functionalities have been abandoned such as parts from circuits of higher speed points of power. Over time this ability to redeploy system components for other uses has become more and more difficult as parts are used up, and the incentive to make parts available has resulted in having to operate the bridge at a slower speed.

6.6.3 Previous Report Recommendations

Previous reports have noted (correctly) that it is theoretically possible to continue repairing this system indefinitely. It has also been noted that this approach will not be cost effective for long into the future. Parsons Brinckerhoff stated in their March 2004 report, the bridge electrical systems are past due for complete replacement and upgrade, and noted that projects of similar scope in the Seattle area have been costing between $1.2 – 2.0 million.

6.6.4 Review Comments and Recommendations

We would concur that the bridge electrical systems are past due for complete replacement.

7. FENDERING SYSTEM

7. TIMBER FENDER SYSTEM

7.1 DESCRIPTION

The fendering system is made up of timber piles and walers. The timber fendering system is positioned to protect the bascule piers from vessels impacting the piers.

7.2 OBSERVATIONS/CONDITION ASSESSMENT

(Relevant Reports: 5 and 9)

Overall condition is fair. A cursory inspection of the condition of the timber fendering system was performed. Significant biological deterioration in the form of fungal decay or dry rot was noted in the tops of the piles. The lower portions of the piles are essentially sound and capable of serving the design function.

7.3 PREVIOUS REPORT RECOMMENDATIONS

The existing reports recommend including the fendering system in the ongoing inspections, but do not recommend that any maintenance be done currently.

7.4 REVIEW COMMENTS AND RECOMMENDATIONS

It is recommended that the fendering system be included in all future inspections. The timber system is showing signs of deterioration, with the tops of the piles in the dolphins having been partially crushed by the cable wrappings. The timber fender system does not appear to have lost the ability to function as required.

8. REHABILITATION/REPLACEMENT ALTERNATIVES

8. ESTIMATED COSTS FOR REPORT RECOMMENDATIONS

8.1 LOAD TEST OF APPROACHES

Estimated Cost: $150,000 (2006 dollars)

The load test of the approaches would consist of the following tasks:

- The bridge structural members are instrumented with re-useable strain sensors.
- A loaded truck crosses the bridge at crawl speed and data is recorded.
- After the truck makes several passes, the instrumentation is removed.
- The data is then "calibrated" with a computer model of the bridge.
- Once this model matches information from the actual structure, rating loads or permit overloads are applied to the computer model and rating values are determined.
- The County would be provided with a report and recommended load ratings.

8.2 STRENGTHENING OF STRUCTURE

Report No. 11 developed conceptual recommendations for a "no collapse" criteria. The criteria assume that for a moderate seismic event (190-year earthquake) the bridge will suffer enough damage to be permanently closed, but the structure will not collapse.

The construction costs are taken from Report No. 11. Costs are in 2006 dollars and do not include design, permitting, contingencies, or mobilization.

Strengthening Location	Construction Cost
Bascule Superstructure	$82,000
Bascule Substructure	$325,000
Truss Approach Superstructure	$130,000
Truss Approach Substructure	$370,000
Concrete Approaches	$255,000
Counterweight Restrainers	$165,000
Total:	$1,327,000

8.3 COMPREHENSIVE SURVEY

Estimated Cost: $80,000 (2006 dollars)

The comprehensive survey would include an overall survey of the entire bridge structure. The goal of the survey would be to capture the entire structure in a single survey in order to be able to determine the interrelationship and position of all of the various bridge components.

8.4 Concrete Coring

Estimated Cost: $120,000 (2006 dollars)

The concrete coring evaluation will consist of taking sample cores from each bascule pier to depths much deeper than previously sampled. The cores will be analyzed to determine the depth of deteriorated concrete and the location of structurally sound concrete.

9. VULNERABILITY COMPARISON

9.1 TASK SCOPE AND DELIVERABLES

KPFF was asked to contact the operators of three other bridges in the Seattle area carrying more than 20,000 vehicles per day, and obtain available information on sufficiency ratings. This information is used to compare the relative vulnerability of the South Park Bridge to other similar bridges in the Seattle area.

9.2 SUFFICIENCY RATINGS OF SELECT BASCULE BRIDGES

Sufficiency rating as defined in the Washington State Bridge Inspection Manual as:

> *"...the basis for establishing eligibility and priority for replacement or rehabilitation of bridges with Federal Highway Bridge Replacement and Rehabilitation Program (HBRPRP) funds. The sufficiency rating is a numeric value, which indicates a bridge's relative ability to serve its intended purpose. The value ranges from 100 (a bridge in new condition) to 0 (a bridge incapable of carrying traffic). The sufficiency rating is the summation of four calculated values: Structural Adequacy and Safety, Serviceability and Functional Obsolescence, Essentiality for Public Use, and Special Reductions."*

KPFF received detailed information from the Seattle Department of Transportation (SDOT) regarding three bascule bridges operated by SDOT. Sufficiency ratings of the Ballard, Fremont, and University Bridges were used for comparison with that of the South Park Bridge.

The three SDOT bridges were constructed between 1917 and 1919. The South Park Bridge was constructed in between 1929 and 1931. The three SDOT bridges use a fixed horizontal trunnion to rotate the bascule leaves. The South Park Bridge uses the comparatively rare Schertzer rolling lift mechanism to rotate the leaves. All three SDOT bridges have been through a seismic upgrade and retrofit program.

Please refer to Table 8.1 for a comparison of load ratings and sufficiency ratings for the bridges. The Fremont Bridge currently has a load rating below 1.0 for the approach spans, however, the approaches are currently being rebuilt.

9.3 DISCUSSION OF RESULTS

Table 8.1 reveals a substantially lower sufficiency rating for the South Park Bridge when compared to the three Seattle area bridges. The South Park Bridge is more vulnerable to closure than the three Seattle area bridges for the following probable reasons: Bascule pier movement, serviceability after seismic events, and ongoing concrete degradation. Each of these topics is addressed individually, below.

9.3.1 Bascule Pier Movement

SDOT has experienced no appreciable relative movement of their bascule leaves and piers when compared to the South Park Bridge, therefore, has historically only performed standard maintenance on the bascule leaves and piers. As noted elsewhere in this report, the ongoing bascule pier rotations and the resulting reduction of the center break gap being experienced at the South Park Bridge have led King County to continually adjust the clearance and fit of the interlocking jaw locks between the two bascule leaves.

There is less than an inch remaining before the center break gap between the bascule leaves will close on the South Park Bridge. The bridge can no longer be shimmed before affecting the pitch of the leaves, and any adjustments at bridge midspan (at the center break) will involve removal of structural portions of the leaves. An in-depth discussion of the bascule leaf adjustments is provided in Section 9.0, Risk Analysis, of this report.

9.3.2 Serviceability After Seismic Events

The South Park Bridge foundations are in alluvium. As described in several of the reports reviewed, the foundations and surrounding soils at the South Park site are subject to liquefaction and lateral spreading after an earthquake. The three Seattle bridges are founded in materials less prone to liquefaction and lateral spreading. The three SDOT area bridges were part of a citywide seismic upgrade program, which was completed in 2000. All three bridges had been seismically upgraded prior to the 2001 Nisqually earthquake.

9.3.3 Ongoing Concrete Degradation

SDOT has not identified similar degradation on the bascule piers for their bridges that King County is experiencing on the South Park Bridge. To date, SDOT has provided only minor patching and maintenance to the bascule piers.

The original timber approaches for both the Ballard and University bridges have been replaced. The current approach structures, like their associated bascule piers, are in good condition and are not experiencing degradation. However, the South Park Bridge continues to experience widespread deterioration of the reinforced concrete portions of the bridge. Concrete deterioration is especially severe on the approach spans where reinforcing steel is now exposed and is suffering material loss as a result of corrosion. Concrete deterioration is also prominent on the bascule piers.

Table 8.1 Vulnerability Comparison	Bridge			
	South Park Constructed 1931	Ballard Constructed 1917	Fremont Constructed 1917	University Constructed 1919
Bascule Load Rating[1]	1.1	1.21	1.11	1.35
Year Rated	1996	1990	1996	1996
Approach Span Load Rating	1.0	1.0	0.8	1.28
Year Rated	1994	1990	1999	2003
Sufficiency Rating (Overall)	4.0	49.2	55.9	64.9
(Approaches)		31.4	13	47
Bascule Pier Stability	Marginal	Stable	Stable	Stable
Life Expectancy	10-12 Years	> 15 Years (Assumed)	> 15 Years (Assumed)	> 15 Years (Assumed)
Recent Work	Spall repairs, bascule span material removal, misc. mechanical rehabilitation Rail repairs Moveable span alignment repairs Misc repairs following 2001 Nisqually earthquake	Seismic retrofit (partial), grating replacement and electrical/mechanical replacement, 1994	Seismic retrofit (partial) and grating replacement in 1998. Full Electrical / Mechanical replacement 2007. Approaches being rebuilt, 2005 to 2007	Electrical/mechanical replacement 1989. Seismic Retrofit (level A) and grating replacement in 1997.

Notes:

1. The load rating factors are for HS20 (operating)

9.4 CONCLUSIONS

In conclusion, the South Park Bridge does not compare favorably with the three SDOT bridges.

The differences in sufficiency ratings as shown in Table 8.1 are primarily the result of the advanced state of concrete and reinforcing steel deterioration in the approach spans, the movements of the bascule piers that occur over time and cause closure of the center break gap at midspan, and to some degree, the observed deterioration of the concrete in the bascule piers. In addition, the alluvium beneath the South Park Bridge pier foundations is more subject to liquefaction and lateral movement during an earthquake than the subsurface soils supporting the three SDOT bridges. Under current conditions, the South Park Bridge is more vulnerable to damage during an earthquake than are the three SDOT bridges.

The three SDOT bridges have had the benefit of both substantial upgrading and retrofitting along with regular maintenance. All three SDOT bridges have had their approach spans replaced, or are in the process of doing so. Conversely, the South Park Bridge has had the benefit of regular maintenance, but has not been upgraded or retrofitted. Rehabilitating or replacing only portions of the South Park Bridge are not economically or structurally reasonable. However, King County has completed many modifications of the bascule leaves in an effort to correct for the continuing movements of the bascule piers.

10. RISK ANALYSIS

10.1 TASK SCOPE AND DELIVERABLES

A risk analysis has been completed to ascertain the risks associated with continued operation of the South Park Bridge. Based upon the age of the structure and its present condition, a number of foreseeable events pose a risk for continued use of the bridge. Examples of risk producing events include, but may not be limited to: earthquake, foundation movements, structural deterioration, and mechanical or electrical failures, among others.

Engineering reports have been written over recent years that summarize the bridge condition and its vulnerability to closure. The vast amount of data contained in these reports has not been synthesized sufficiently to quantify the risk that King County has incurred while continuing to operate the bridge.

This risk analysis addresses King County's current interest in developing answers to two important questions.

1. What risks are associated with continued use of the bridge?
2. At what level of risk should the County close the bridge?

King County has defined the expression "continued use of the bridge" as the ability to use and operate the bridge under current traffic volumes without the necessity to substantially upgrade or repair the structure. The "ability to operate" includes normal opening and closure of the bascule spans.

10.2 TASK DELIVERABLES

King County identified specific task items to be included in the risk analysis work. Tasks specific to the risk analysis include the following:

- Examine and identify a comprehensive set of risk factors for the existing bridge.
- Ascertain peak ground accelerations for several seismic events having 10 to 50-year return period, and identify the probable effects such events may have on the bridge.
- Based on the full range of identified risk factors, identify any failure scenarios that would be impractical to repair, leading to bridge closure.
- Based on the Consultant's professional judgment, determine and explain the relationships between level of risk, time, and applicable risk thresholds.
- Based on the comparative analysis of risk versus time, determine at which level of risk the bridge should be closed and provide reasoning for the determination.

10.3 RISK ANALYSIS METHODOLOGY AND APPROACH

There are three distinct types of risk that imperil any plan for continued operation of the South Park Bridge. The first consists of continuing to operate the bridge until a catastrophic event occurs that will force the County to invest funds into bridge repair that are a high proportion of the bridge replacement cost. Doing so creates a condition where the investment in the repair provides a repaired bridge, but one with a very short service life relative to that of a replacement bridge. This type of risk is generally associated with economic risk, and is defined in this analysis as cost-based risk.

A second type of risk is losing the operational functions of a movable span bridge. This type of risk also applies to the South Park Bridge, and is defined as Operational Risk in this analysis.

There are also safety risks associated with continued operation of the South Park Bridge in a seismically active geologic zone. Portions of the bridge approach spans are particularly vulnerable to seismic loading and may collapse during a moderate earthquake. In this analysis, this is defined as Safety Risk.

10.3.1 Cost-Based Risk

Cost-based risk is calculated in terms of monetary units (dollars). As defined, these dollars represent the risk being incurred by King County by deferring either substantial rehabilitation or replacement of an aging structure that is vulnerable to sudden closure or partial collapse. Except in the case of annual maintenance, these dollars should not be confused with funds that are actually drawn from the County budget.

The cost-based risk approach used in this analysis is similar to that used by the WSDOT. The WSDOT risk assessment tool is used during the project planning and development stage and identifies cost implications for a number of events that may affect the final project cost during project implementation, and assigns probabilities to these events. The resulting matrix of values provides WSDOT with a range of probable cost outcomes, and also allows for visibility to make project plan changes during the design and construction phases.

A comprehensive list of probable events that may cause discernable consequences, which in turn may lead to closure of the bridge, has been identified. These discernable consequences have been defined as risk factors. In establishing this list, it was prudent to limit the list to those events that are most likely to occur within the time frame for this analysis, taken as the next 20 years by mutual agreement with King County. Examples of two such events include the continued settlement and lateral movement of the bascule spans leading to an inability to open and close the bridge, the occurrence of a significant seismic event that may cause severe structural damage to the approach spans, and additional bascule settlement and lateral movement. The subject of bascule pier settlement and lateral movement is a very critical issue regarding the estimated time remaining until bridge closure. This issue is discussed in considerable detail later in this section of the report, but can be summarized here as follows.

The actual cause or causes for the observed bascule pier movements has not been established. However, two apparent causes are credible. The first cause is continuing rotation of the bascule piers resulting from gradual settlement of the foundation soils, and the second cause is sudden discrete rotations associated with seismic events. Based upon the documentation provided by King County, it can be speculated that a small portion of the accumulated rotation is the result of settlement of the foundation soils and a larger portion is the result of a series of seismic events (1949, 1965, and 2001) that have occurred since the bridge was constructed. It is critical to the risk analysis process that both of these credible events are considered when assessing the risk of continued use of the bridge, because the probabilities of these two separate events are not equal over time. As discussed later in this section, both events are included in the risk analysis computations. However, additional verification studies are needed to better quantify and verify the accuracy of the assumed contributions of each event to the total time dependent predicted rotation of the bascules and, therefore, the estimated remaining time to bridge closure.

An example of a lower probability event may include a vehicle fire on the bridge that would likely cause little, if any, bridge damage and was, therefore, not included in the present risk analysis. A complete list of hazardous events included in the risk analysis is provided in Attachment 1 Risk Analysis Summary at the end of this section.

It is important to note that the risk analysis incorporates a series of independent events, each having a discernable consequence that can lead to possible bridge closure. An independent event is defined as an event that can occur without dependency on other risk creating events.

In an effort to determine a relative level of overall risk for the full range of independent events, a matrix was developed that compares the cost consequence (or loss) for each identified risk that has been multiplied by the probability of that event occurring within any time span under study. The product of the event probability, multiplied by the consequential cost associated with occurrence of the event, is defined as the associated cost-based risk. For those events that are driven by ongoing deterioration of the structure, both the probability of event occurrence and the cost consequence of the actual event increase with the passage of time. For other events, such as earthquake, the probability of occurrence is constant with time. One fundamental increase in consequential cost with time is the result of escalation of repair costs for all events.

Consequential costs resulting from the occurrence of an event are defined as those that are directly related to repair and/or replacement of portions of the bridge structure and mechanical systems. Consequential costs include design and management costs and construction costs. Although there are other indirect costs associated with the need to repair the bridge following a significant event (incidental costs), these are not included in the scope of this risk analysis. Examples of related incidental costs include traffic re-routing, sales losses to local business owners, and associated costs resulting from increased commute times.

There is one important consequential cost that has been included in this risk analysis that is not directly related to bridge upgrade or repair. This is the annual operation and maintenance cost for the bridge, which has averaged $543,000 for years 1995 through 2003. Similar costs will continue to be incurred as long as the bridge remains open and, therefore, have been included in the cost-based risk analysis. The probability assigned to the occurrence of maintenance cost is 1.0 (certainty).

The tabulated summary of the risk analysis is provided in Attachment 1. It can be seen by adding the associated cost-based risk for each independent event and summing the events that are probable within a given time period, the total cost-based risk can be determined for any time period studied. The total cost-based risk is then compared to the threshold cost for bridge replacement to provide King County with a measure of the relative risk being incurred. The cost-based risk approach provides a way to define and quantify the relative economic risk associated with the occurrence of an event. This approach is valid if one assumes that the bridge is not scheduled for major repair or upgrading during the period of the risk analysis study. By assuming that the bridge is not repaired, it is possible to use the cost-based risk values to predict when the bridge should be considered for replacement.

10.3.2 Time Dependency of Cost-Based Risk

The time dependency of cost-based risk depends in part on the nature of the type of event that is anticipated. Two event types have been categorized for this risk analysis.

- Discrete Events – These events are events that are continually ongoing with time. Examples of such events include deterioration of the reinforced concrete structures and wear and fatigue of bascule mechanical systems, among others. Also, movements and lateral displacement of the bascule piers may be partially attributed to minor settlement and rotation of the entire bascule pier foundations. Trends in the occurrence of these events were established by reviewing data from previous studies and extrapolating this data to the future. Without remediation, the ultimate resolution of a discrete event may be structural failure, operational failures of the bascule lifting system, or reduced load capacity of structural members that in turn may require reducing the load rating and sufficiency of the bridge. Therefore, the time dependency of cost-based risk for a discrete event is dependent on anticipated time dependent trends and the repair cost escalation that occurs with the passage of time. Where event trends are associated with maintenance they are continued with a probability of 1.0 (100 percent probability of occurrence) for all time periods studied. Where trends need to be inferred, such as for the mechanical systems, probabilities are established based on engineering judgment and experience.

- Random Events – Random events are statistically probable events that can cause significant damage to the bridge. These events occur suddenly and are not characterized by a continuing behavior trend. Typical examples include earthquakes and marine vessel collisions. It can be speculated that the rotation of the bascule piers may be largely attributable to seismic events and, therefore, may be classified as random events. Random events can only be described in terms of probability of occurrence within any given time period.

The initial consequential costs are presented in 2005 dollars. All of the initial consequential costs presented in Attachment 1 "Risk Analysis Summary" are calculated by extracting estimated costs arising for each event as found in the data included in study reports completed by others and provided by King County. These initial costs were then escalated to the 2005 timeframe. For outlying years, all consequential costs are also adjusted based on an assumed annual price escalation of 3 percent. This escalation value was taken from the average annual value for the Consumer Price Index for the Pierce County/Tacoma area as of September 2005.

An additional adjustment has been made to the consequential costs for the seismic events during outlying years. It is recognized that during the periods between seismic events, the structural condition of the bridge is expected to continue to deteriorate. Therefore, for the seismic events, the outlying year consequential costs have been increased by 2 percent per year to reflect the anticipated increase in consequential costs. The value used is based on engineering judgment.

10.3.3 Risk Analysis Methodology Limitations

The current risk analysis contains several limitations. They are described in the following:

1. Analysis Focuses on Bridge Closure

   As defined in the project scope of work, this risk analysis focuses on the probability of occurrence of one or more events that may lead to closure of the bridge. Scenarios can be developed that include extending the useful life of the bridge by a series of upgrades and repairs. Such scenarios are beyond the scope of this risk analysis. For this analysis, the bridge is assumed to receive only continued maintenance and minor repairs throughout the assumed study period. King County has on file several study reports that address analyses of bridge life cycle costs for various rehabilitation or replacement scenarios.

2. Analysis Excludes Bridge De-Rating Options

   One method that can be used to extend the service life of bridges is to restrict the use of a bridge to vehicles of higher axle weights. This process is often referred to as "de-rating" the bridge. Previous study reports, as recent as 1994, noted that the condition of the concrete slabs in the approach spans was questionable and suggested that consideration be given to repair in order to prevent a decline in load rating. These specific repairs have yet to be made, and King County has stated that there is no immediate plan to operate the bridge in a de-rated status. Therefore, bridge de-rating options are excluded from this analysis.

3. Analysis Excludes Incidental Costs

   The risk analysis matrix defines the forecasted risk as Associated Cost-Base Risk. The units contained in these results are the probability of event occurrence multiplied by the estimated costs to repair the bridge. Additional costs that accrue as a result of issues that are not directly related to repair design and construction are called indirect or incidental costs and are not included in this risk analysis. Examples of incidental costs include those associated with re-routing traffic during bridge reconstruction and loss of local business income.

4. Analysis Relies on a Limited Database

   The methodology employed to develop much of the data used as input to the risk analysis relied on data that was extracted from previous study reports. Examples of some of the data used includes data and information regarding the continuing movements of the bascule spans, the laboratory test reports on deteriorating concrete, and data related to bascule and approach span foundation settlement trends. Sections 4 through 7 of this report describe the database used for this study and analysis, and provide a discussion of the apparent database limitations. In a general sense, it was beyond the scope of this project to expand or even replicate any of the data provided by the previous studies. A notable exception to this limitation is that site-specific seismic risk parameters were developed independent of the work done in previous studies. These parameters are provided in Section 3 of this report.

10.3.4 Risk Analysis Parameters

There are three primary parameters that shape and influence the risk analysis results. They are: the analysis time frame, event probabilities of occurrence, and the criteria for bridge closure. The relationships between these parameters are defined in the following:

1. Analysis Time Frame

   Studies that address the probability of future events must include a limitation on the extent of the time frame studied. In theory, if the time frame is unlimited, the probability of occurrence for each event would equal 1.0 (certainty of occurrence), and the study effort would essentially be moot. Establishing a reasonable time frame provides realism to the effort and enhances confidence in the meaningfulness of the results.

   In addition, because the project scope requires that the risk analysis include a definition of risk as a function of time (time dependency) and because for this analysis, risk is defined in relation to consequential costs that may be incurred, it is also essential to establish a time frame for the analysis. Doing so allows the analysis to include the effects of escalation on the anticipated consequential costs.

   The South Park Bridge was constructed approximately 75-years ago; compared to similar bridges in the Seattle area it has a low sufficiency rating. Several deterioration mechanisms are continuing to occur at the bridge, and the bridge is already in an advanced state of deterioration. The age of this structure and its advanced state of deterioration suggest that the risk analysis should be based on a realistically short time frame. For these reasons, the bulk of this study addresses the probability of events and their consequences for a period of 20-years beginning in 2005. At the end of that time period, the bridge will be approximately 100 years old.

2. Event Probability of Occurrence

   It was previously defined that two event types are included in the risk analysis: discrete and random. Occurrence probabilities for discrete events (such as bridge deterioration) are based on observed trends in past bridge condition and performance. Occurrence probabilities for random events (such as earthquake) are based on statistical analysis of similar events that have occurred in the Puget Sound region. The risk analysis method employed in this study recognizes the time dependency of event occurrence.

3. Criteria for Bridge Closure

   In order to estimate the time remaining to bridge closure, it is not sufficient to simply project event occurrences and their consequential costs. It is also necessary to define what criteria are used to require bridge closure. King County defines the end of the useful service life of the bridge as follows:

   - When the movable span of the bridge cannot be operated.
   - When the bridge can no longer safely support a legal vehicle load.
   - When the maintenance and operations costs exceed what can be included in future operating budgets.
   - When the safe load carrying capacity of the bridge can no longer be determined.

The American Association of State Highway and Transportation Officials (AASHTO) also require that "bridges not capable of carrying a minimum gross live load weight of three tons must be closed."

In addition, the bridge should be considered for closure when the magnitude of the cost-based risk as calculated in this risk analysis equals 70 percent of the replacement cost of the bridge. The rationale for this suggestion is related to issues regarding federal funding for bridge replacement projects.

With regard to this criterion for bridge closure, the evaluation of this issue is already known. In 2003, a previous study stated that the estimated rehabilitation cost for the bridge was $63,930,000, and the bridge replacement cost was estimated as $77,334,000. Equivalent values for the year 2005 are $67,824,000 and $82,000,000 respectively, indicating that the 2005 rehabilitation cost is approximately 82 percent of the anticipated replacement cost. In essence, the current rehabilitation cost is larger than 70 percent of the current replacement cost, meaning that rehabilitation is no longer a viable option if outside funding sources are needed. Therefore, for the South Park Bridge, triggering one of the mandatory bridge closure criteria will signal the requirement for replacement.

The actual risk being assumed by King County is analogous to continuing to invest significant repair costs in an aging asset that is nearly at the end of its anticipated maximum service life. When the money spent on repairs rivals the magnitude of the purchase price for a new asset, the investor still owns an old asset, but at nearly the same cost as investing in a newer asset with a much longer remaining service life. The risk analysis completed for the South Park Bridge embodies this concept. Substantial investments made for rehabilitating this bridge are no longer sound and the concept of bridge closure must be equated with bridge replacement.

Therefore, bridge closure criteria must focus on the "risk" assumed by not rehabilitating the bridge while continuing to use the bridge, rather than financial considerations related to the potential benefit of rehabilitation versus replacement. Because of this key issue, this cost-based risk analysis addresses the risk being assumed by continuing to use the bridge in its current condition.

10.3.5 Cost-Based Risk Analysis Results

A total of 25 events have been analyzed as possible contributors to the future closure of the South Park Bridge. Of these, 17 are categorized as discrete events and 8 are random events. A complete list of these events and their cost-based risk for five time periods are compiled in Attachment 1 "Risk Analysis Summary." The five time periods, beginning in 2005, are as follows: Year(s) 1, 5, 10, 20, and 50. The consequence for each event is also provided in the tabulated risk analysis summary, along with the initial consequential costs in 2005 dollars. The initial consequential cost column is followed by a series of columns (one for each of the five study time periods) that contain the probability of occurrence for each event within that specific time period. The last five columns of the cost-based risk analysis summary contain the Annual Cost-Based Risk (the product of the event probability times the escalated value of the consequential cost) for each event. It should be noted that the values in these last four columns are presented as costs for each year in the period.

It should be emphasized here that for events that have a probability of occurrence of less than 1.0 (100 percent probable), the Cost-based risk values do not equate to the costs that the County will actually incur in the time periods studied. Rather, the cost-based risk tabulation for these events provides a means to define and quantify the relative risk associated with the occurrence of independent, yet probable events that can occur in any time period. By adding the cost-based risk values for each independent event and comparing them to the bridge replacement cost, it is possible to estimate the relative level of overall risk that is being assumed by King County while continuing to operate and use the bridge during any period of interest. Examples of such events include continuing movements of the bascule piers, concrete and reinforcing deterioration in structural members, and seismic events.

Conversely, for events that have a probability of occurrence of 1.0 (certainty), the cost-based risk values do represent costs that will occur within a given time period. An example of such events includes recurring annual maintenance.

It should also be noted that although the analysis includes five probable seismic events of interest, only the maximum cost-based risk value is used in a given time period when summing the risk values in the table. The rationale for this is that the table is developed to determine the single seismic event having the largest numerical value of the product of the event probability and the consequential cost.

The remaining task prior to using these results is to determine the estimated replacement cost of the bridge for each of the five periods studied. The replacement cost for the bascule bridge option was estimated to be $77,334,000 in 2003 dollars. This places the replacement cost in 2005 dollars at approximately $82,044,000. The estimated replacement costs for the remaining study periods are Year 5: $95,111,000, Year 10: $110,260,000, and Year 20: $148,200,000 and Year 50: $359,673.000.

As previously defined, when the magnitude of the cost-based risk equals or exceeds 70 percent of the replacement cost in any time period, the bridge should be subject to closure. Therefore, the Risk Analysis Summary provided in Attachment 1 defines this 70 percent value as the "Threshold Cost." Threshold Costs are as follows:

Year 1	$57,431,000
Year 5	$66,578,000
Year 10	$77,182,000
Year 20	$103,727,000
Year 50	$251,771,000

10.4 Cost-Based Risk Event Analysis

10.4.1 Bascule Pier Piling Degradation and Loss of Capacity

This category of events applies to Event Nos. 1 and 2. This event category concerns the existing timber piling that currently support the bascule piers and the foundations at both approach spans. Previous study reports do not include any inspection results or analyses related to the condition and load capacity of the timber piles supporting the bascule piers. However, previous reported information indicates that repairs have been made to some piling supporting the approach spans. For the most part, the timber piles were installed below the Duwamish River low waterline. Also, for the bascule piers, the upper portions of the piles are encased in concrete.

The state of knowledge regarding similar pile installations of like vintage was reviewed and that review found no compelling reason to suspect that the bascule pier piles are suffering serious degradation at this time. Technical references on this subject suggest that round timber piling installed in the ground below mudline and below the water table may last for very extended periods of time. When the untreated timber piles in an 80-year old Erie Canal structure located in Waterford, New York, were extracted in 1996, they were found to be in virtually pristine condition. Similar reports on other structures are evident in the literature.

The timber piles can be expected to function well for typical foundation loading. However, there may be a link between the observed bascule pier rotation issue and the performance of the timber piling under seismic loading. This potential link is discussed later in this report. For these reasons, the probability for piling failure under loading other than seismic, and the estimated cost-based risk for piling repair were estimated at low values in the risk analysis summary.

10.4.2 Approach Pier and Steel Truss Pier Piling

This event category applies to Event No. 5. During the 1970s, the north approach pier was inspected and one of the key findings was that some of the timber support piling were seriously distressed and required repair. The cause for the distress (overload, deterioration, or other) was not identified in the information provided by King County. Suggested repairs included excavating around the base of the pier and encapsulating the tops of the damaged piles with concrete. King County has indicated that these repairs were completed.
No additional information exists regarding other approach pier piling, and based upon the information provided by King County, it is unlikely that the repaired piles were inspected in the years following repair.

The records provided by King County do not contain any information that addresses either inspection or repair of the timber piles that support the steel truss piers.

The previous damage record for some of these pilings, make them vulnerable to failure. For this reason, the probability of failure of the timber piles supporting the approach and truss piers is significantly higher than that for the bascule piers. Entries in the Risk Analysis Summary indicate these differences.

10.4.3 Degradation of Reinforced Concrete

This category of events applies to Event Nos. 4, 6, 7, and 8. Previous reported investigations of the concrete condition at the bridge have identified numerous areas where the concrete is heavily distressed. Several reports contain information that is highly speculative and the conclusions drawn from these speculations may not be overly reliable. The issues regarding the reliability of conclusions drawn from these studies are discussed in detail in Sections 4, 5, and 6 of this report.

The existing data related to structural concrete deterioration has been reviewed. In addition, a brief visual inspection of the bridge condition was made and a review of the original bridge design drawings was completed. The following summarizes findings related to concrete and structural deterioration issues.

1. Decreasing concrete strength

   It is highly likely that the concern for decreasing concrete strength in the bascule piers is not valid. Section 6 of this report provides a rationale for that conclusion. Therefore, this concern has not been included in the risk analysis.

2. Deterioration of concrete

   The concern that there are extensive areas where the concrete has deteriorated and continues to deteriorate is fact. It is also true that the testing completed on the concrete was not directed to determine the actual cause or causes for the observed deterioration. Therefore, specific repair remedies cannot be proposed with any assurance that they would be successful. Therefore, because the specific deterioration mechanism(s) are unknown, it is not possible to accurately predict the remaining life of these concrete structures. The evaluation of the concrete deterioration issue was based only on the available information found in previous studies, supplemented by observations made at the bridge site. Based on engineering judgment, it has been concluded that the causes for the observed deterioration are as follows:

   - For the approach span decks, freeze/thaw damage is the most probable cause.
   - For the approach span superstructure, a combination of low concrete cover over reinforcing steel, freeze/thaw damage, reinforcing corrosion, and alkali-silica reactivity in the concrete are likely causes.
   - For the bascule piers, elevated chloride ion content and possible ASR or DEF in the concrete matrix are possible causes for the observed distress. However, there has not been any confirmation testing to identify that either ASR or DEF are occurring in the concrete bascule piers.

3. Vertical cracks in bascule piers

   Wide vertical cracks exist in the bascule piers, and King County modified the bascule piers in 1982 by adding post-tensioning across the cracks. The cracks are large (wide), and indicate that any embedded reinforcement located in the vicinity of these cracks have either fractured or have been loaded beyond its' yield strength. Specific causes for the cracks have not been identified.

The importance of each of these structural and concrete materials issues varies with the structure they act on. Therefore, the influence on "risk" for continuing use of the bridge also varies, resulting in variable influence on the cost-based risk values presented in the risk analysis summary. The following explains these issues.

- Freeze/thaw damage of the approach span decks has progressed to a severity that, when combined with other deterioration mechanisms, can lead to bridge closure. Substantial repairs of affected areas are anticipated to be needed in approximately five years, and this is reflected in the high probabilities associated with this event that are listed in the risk analysis summary.
- The combination of possible damage mechanisms on the approach spans combined with the advanced state of deterioration influence the risk analysis by establishing high probability values for aggressive deterioration and high consequential cost values. In essence, the advanced state of

concrete deterioration is having a marked effect on the embedded reinforcing steel, giving rise to concerns for reduced load carrying capacity in the approach spans.

- The concrete deterioration occurring at the bascule piers has caused wide cracks to open in the lightly reinforced structure and is causing large amounts of spalling and cracking of the piers. Nonetheless, the bascule piers are very large gravity foundations supported on timber piles that are not likely compromised. The bascule piers are very lightly reinforced, making the effect of concrete spalling far less critical than it would be for more conventional reinforced concrete members.
  However, the large cracks that have developed in the lightly reinforced piers are a cause for concern. Also, the bascule pier concrete has been tested and found to contain high chloride ion contents, some sufficiently high to readily cause corrosion in the reinforcing steel. However, because the piers are so lightly reinforced, and because no data exists that indicates the remaining reinforcing steel is sufficiently corroded to reduce the load capacity of the piers, the consequential cost of this event is low, although the probability of this event is very high in outlying years.
  In general, the overall influence of concrete material deterioration on the bascule piers presents a low probability of failure within the study period. However, the large cracks that have opened present a significant concern for the piers. The probabilities listed in the Risk Analysis Summary reflect these combined deterioration modes.

10.4.4 Bascule Pier Movements

This category relates to Event Nos. 3 and 20. Considerable data exists that demonstrates that the bascule piers may be rotating and settling, causing interferences at the center break joint between the two bascule leafs. This issue is very critical and its effect has a direct bearing on the estimated remaining time to bridge closure. The interferences between the two bascule leaves have been offset by King County physically shimming the bascule leaves and modifying the interface between the leaves at bridge midspan. However, it is predicted that additional bascule movements will eventually cause the leaves to interfere and similar remedies will not remove the interference unless the bascule leaves are modified. At that time, it will no longer be possible to operate the bascule span, and the bridge will be subject to closure.

The actual cause or causes for the observed bascule pier movements is uncertain. However, two potential causes are credible, those being a continuing rotation of the bascule piers resulting from gradual settlement of the foundation soils and sudden discrete rotations that may be associated with seismic events. Based on the documentation provided by King County, it can be speculated that a small portion of the accumulated rotation is the result of settlement of the foundation soils and a larger portion is the result of a series of seismic events that have occurred since the bridge was constructed. In the absence of a documented cause and effect relationship that explains the bascule leaf interference problems, it was necessary to hypothesize a reasonable relationship in order to complete this risk analysis. The hypothesis described in the following is based on review of the existing related documents and engineering judgment.

The individual movements of the two bascules can be described as rigid body rotation of the bascule structures about two axes. Rotations occur about a horizontal axis that cause the bascule leaves to close the gap at bridge midspan and also about a vertical axis that cause the bascule leaves to skew and misalign the roadway centerline. The combined set of rotations may be caused by a combination of issues. The first is that the center of gravity of each bascule is located off center on the bascule pier footprint with the misalignment placing the leaf weight toward the waterway centerline. The second issue is that the data available for studying the bascule rotations suggests that the large majority of the rotations may have occurred either during, or soon after, seismic events. However, the historic trend for the measured rotations suggests that a smaller portion of the accumulated rotation occurs gradually over time. Therefore, it can be speculated that the bascules continue to rotate at a very low rate, and random seismic events cause large additions to the total rotation.

Two types of data are available for studying the bascule movements, and both have been reviewed in an effort to develop movement trends that can be used to predict the remaining time to bridge closure. King County instrumented the two bascule piers early in the 1990s and has attempted to gather data on bascule movements to predict trends. This data has been reviewed, and it has been determined that the data is not readily useful for quantifying past movement trends and for predicting future trends.

Rather than request additional inclinometer data for review, a second approach was used to ascertain trends for bascule rotation. To accomplish this, the data provided by King County that documents King County activities involving efforts to compensate for bascule leaf interferences at the center break was reviewed in an effort to establish movement trends.

Two techniques have been used by King County to reset the bascule span alignment to mitigate interferences that may result in the inability to properly open and close the bridge. Recent efforts have been ongoing since 1996; however, some evidence suggests that a modification to the center break may have been made in 1950, or earlier.

The first technique to avoid leaf interference at the center break has been to add shims between the leaf counterweight arm and the fixed rotation stop guide to stop the bascule rotation short of a full closure. By adding the shims, the cantilevered leaves are slightly inclined toward the center break along with a slight drop at the tail breaks to compensate for the shimmed pitch of the leaves.

The second technique used by King County has been to shorten and reshape the interlocking steel "teeth" that fill the gap between the two bascule leaves. Doing so compensates for some of the interferences at the leaf tips due to bascule pier rotation about the horizontal and vertical axes.

The information provided by King County on this issue leads to the conclusion that a large majority of the bascule rotation that has taken place, since the bridge was constructed, has been the result of rigid body rotation of the bascules following seismic events. In all probability, a lesser amount of rotation has been caused by gradual "creep" of the foundations caused primarily by the eccentric loading of the bascules on the bascule piers.

A detailed description of the rationale for this conclusion is provided in Section 6.3 of this report. The current (2005) gap that exists between the two bascule leafs at the bridge center break is approximately 1-inch. It has concluded that the average gap change that occurs during a typical significant seismic event is approximately 3/4-inch. Three significant seismic events have occurred since the bridge was constructed, those being in 1949, 1965, and in 2001. Because there have been a number of iterations involving material from the steel interlocking teeth, further modifications to the teeth cannot be made without compromising the center break joint. Furthermore, installing additional shim packs at the bascule counterweights cannot be used as a viable remedy without creating a discontinuity in the vertical profile of the roadway at the tail breaks of the moveable spans.

Therefore, the bascule leaf system of the South Park Bridge is anticipated to remain operable in its present condition for a limited amount of time. However, given the hypothesized causes for bascule rotation, a seismic event of significant magnitude to cause the bascules to rotate sufficiently to close the center break gap can occur at any time. When this event occurs, it can be forecasted that the bridge lift spans will no longer be operable, and the bridge will be closed out of necessity. Also, the bascules continue to gradually rotate with time and will eventually cause closure of the center break gap even if a significant seismic event does not occur.

The seismic event that is assumed to close the bridge due to bascule rotation has a return period of 190-years (10 percent probability of exceedence in 20-years). This event is similar to the 2001 Nisqually earthquake.

The limited amount of quantitative data available to describe the gradual rotation trend (not influenced by seismic events) of the bascule piers suggests that gradual bascule rotation sufficient to close the bridge may occur within the next 40-years. However, recent measurements taken by the County since 2003 suggest a shorter projected time to bridge closure. The time dependent probabilities for this event are listed in the risk analysis summary under Event 3 where the consequence of this event is to replace the bascule, or bascules, that have rotated. The consequential costs for gradual bascule rotation are provided under Event 3 in the risk analysis spreadsheet.

The probabilities for sudden rotation caused by the 190-year return seismic event are included in the probabilities listed in the risk analysis summary under Event 21. When this event occurs, other seismic related consequences will also occur, and the consequential cost will equal the replacement cost for the bridge. The time dependent probabilities for this event are given in the risk analysis summary sheet.

10.4.5 Mechanical/Electrical System Risks

This category relates to Event Nos. 9 through 16. Potential failure of the mechanical and electrical systems are significant in that a failure that prevents operation of the bascule lift systems will render the bridge closed. Risks associated with the continued operation of the mechanical and electrical systems can be considered as two categories of risk; those associated with continuing degradation and wear of system components, those associated with the occurrence of a major seismic event.

Wear degradation of system components can be estimated, and the useful life of components can be extended somewhat by providing aggressive system maintenance. However, system components generally have a finite life and will eventually require replacement regardless of the level of effort provided for maintenance. Previous condition assessment studies for the mechanical systems were reviewed, and for the most part, when major component replacements have been suggested, King County has completed the work and currently reports that the bridge machinery operates satisfactorily.

The second risk category for the mechanical systems addresses the anticipated response of the mechanical systems in the event of a seismic event having a 190-year return period. Critical components of the mechanical system that are anticipated to require replacement following such an event are racks and pinions on the main bascules. Substantial repair of the rocker/track engagement components should be planned.

Probabilities for replacement of critical mechanical drive components have been assigned (Events 9 through 13) as shown in the risk analysis summary. The assigned probabilities for these events address the consequences that will arise in the event of seismic events. The probabilities for component replacement become significant approximately 10 years hence, and two major components should be scheduled for replacement within 20 years.

Components of the electrical system will eventually reach obsolescence. The probabilities of the need to do so and the consequential costs associated with doing so are reflected in the risk analysis summary.

10.4.6 Seismic Risk

This category relates to Event Nos. 18 through 22. Applying the cost-based risk to random events such as earthquake requires additional definition. At issue is the fact that random events are not precisely predictable over time, and can only be discussed in terms of their statistical probability of occurrence. Therefore, any and all of the earthquake events used in this risk analysis can occur at any time, but their probabilities of doing so vary considerably. It should also be noted that during periods when seismic events do not occur, the bridge continues to degrade as described in the discussions that address discrete events. Therefore, it was necessary to not only determine the probabilities of occurrence for various levels of earthquake likely to occur within the study period of 20 years, but it was also necessary to include the effects of continuing structural degradation during all periods.

The approach taken was as follows:

- Establish a "threshold" seismic event where the threshold is defined as the occurrence of an earthquake of sufficient energy to damage the bridge to the extent that total replacement is necessary.
- Considering the time period selected for this study, determine the basic probabilities for various intensity events based upon probabilistic return periods.
- The cost-based risk for any event having an energy magnitude equal to or greater than the threshold event is computed by multiplying the basic probability of the event times the 2005 bridge *replacement* cost escalated to the future time period as required. Conversely, the cost-based risk for any seismic event having an energy magnitude less than the threshold event is computed by multiplying the basic probability of occurrence times the appropriate 2005 bridge *repair* cost escalated to the future time period.
- In any given time period, the cost-based risk for events below the threshold and those above the threshold are compared, and the maximum value is provided in the risk analysis summary. This is done to assure that events of lower consequential costs, but having higher probabilities of occurrence, are not overlooked in the analysis.
- Because the bridge is assumed to continue to degrade with time, somewhat higher consequential cost values were applied to the events in outlying years. For these events, the periodic consequential costs have been increased by 2 percent annually to account for the continuing time dependent deterioration. This increase is in addition to the 3 percent annual escalation factor previously described.

10.4.7 Vessel Impact Risk

This category relates to Event Nos. 23, 24, and 25. There is no King County record of a vessel impact at the bridge. The timber fender system that protects the main bascule piers was inspected in 1997 and rated to be in fair condition.

The probability of vessel impact sufficient to cause major bridge damage is very low. However, should such an event occur, and the bridge is damaged sufficiently to require closure, it will be imperative to conduct immediate emergency repairs to at least re-open the navigable channel to water traffic. The Duwamish Waterway is considered to be a Navigable Waterway under the jurisdiction of the United States Coast Guard.

In the event that the bridge should become impassable to marine vessels, the Coast Guard has the authority to impose fines on King County for amounts as high as $10,000 per day or mandate the "permanent" opening of the bridge and subsequent removal of all in-water obstructing structures. Although the Coast Guard has the authority to impose these fines, the likelihood of doing so will be small if King County moves quickly and diligently to remove any waterway obstructions that might disrupt in water traffic. For this reason, no allowance was made for imposing these fines in the consequential cost values provided in the Risk Analysis Summary.

The probability of large vessel collision with the piers is low, but has been included in this risk analysis. The probability of large vessel impact at the in-water approach structures supporting piers is extremely low due to the water depth and the effective channel width near the approach spans, but has also been included in this analysis.

10.4.8 Cost-Based Risk Conclusions

The results of the cost-based risk analysis indicate that the total cost-based risk will exceed the Threshold Cost in approximately 11 to 12-13 years. Therefore, the South Park Bridge should be scheduled for closure and replacement prior to the year 2016.

Figure 10.1 is a plot of key information provided in the Risk Analysis Summary spreadsheet. The figure provides a visual representation of the predicted remaining time to bridge closure and, therefore, replacement. The cost-based risk values are plotted overlying a plot of the variation in the threshold cost with respect to time. The intersection point of these two plots occurs within a time period of approximately 12 years beyond 2005. Based on these results, the South Park Bridge should be scheduled for replacement by the year 2016.

The results of the analysis also indicate that the event most likely to lead to closure is the 190-year return seismic event. Specific causes for closure will likely include extensive structural damage to one or both approach spans, and/or additional bascule leaf rotation sufficient to close the center break gap to prevent opening and closing of the bridge.

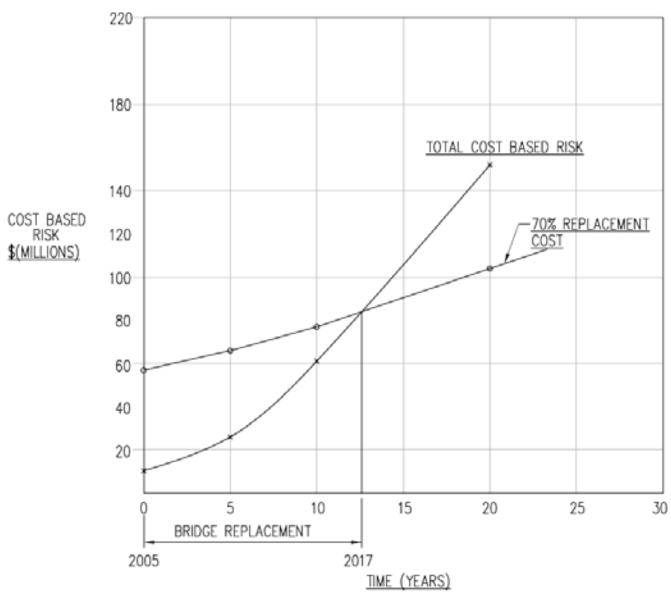


Figure 10.1

ATTACHMENT 1 - RISK ANALYSIS SUMMARY

Notes

1. Bascule Bridge option, 2005 dollars, King Co. Structural Alternatives Study, 2003. 2003 dollars adjusted to 2005 dollars at 3 percent annual escalation.
2. Seismic collateral damage. Probability of occurrence is the same as seismic event – 10 percent exceedence in 25 years.
3. Threshold Cost is defined as the 70 percent of the Replacement cost in 2005 dollars. South Park Bridge Rehabilitation Study, May 2003. Out year costs are escalated at 3 percent per year.
4. Average annual maintenance costs from 'Analysis of Risk and Remaining Life of the South Park Bridge, March 2004. $543,000 in 2004 escalated at 3 percent for one year to 2005.
5. 1993 estimated cost of $250,000 escalated to 2005 at 3 percent per year.
6. 2004 PB estimated cost of $2M escalated to 2005 at 3 percent per year.
7. No existing data for repair cost for EQ-95. Estimated value is based on approximate amount of energy released relative to 'Threshold' event EQ-95 releases approximately 10 percent of the energy in EQ-190.
8. Estimated cost to partially demolish and rehabilitate existing mass concrete pier, 2005 dollars.
9. Only Event 5 or 6 is counted in the Discrete Risk subtotal. These high-probability events result in the same replacement consequence.
10. Seismic based repair costs have been escalated 2 percent per year to account for continued structure degradation in outlying years.
11. Energy released in EQ-50 is approximately 1/100 of 'Threshold' event of EQ-190. Consequential costs are rationed accordingly.
12. No cost estimate available for EQ-20. Estimated repair cost at $500,000.

SOUTH PARK BRIDGE REVIEW
Risk Analysis Summary
(DRAFT)

EVENT NO.	EVENT (or HAZARD)	RISK FACTOR (or CONSEQUENCE)	CONSEQUENTIAL COST $	Probability Event Will Occur in Time Period					Associated Annual Cost-Based Risk					Notes
				Year 0-1	<5 Years	<10 Years	<20 Years	< 50 Years	Year 0-1	<5 Years	<10 Years	<20 Years	< 50 Years	
	'Discrete' (or Ongoing) Risks													
1	Bascule Pier Timber Piling Load Inadequacy (Initial Risk)	**Replace Piers and Foundations**	$34,384,000	1.00%	1.00%	1.00%	1.00%	1.00%	$344,000	$399,000	$462,000	$621,000	$1,507,000	1
2	Bascule Pier Timber Piling Degradation	**Replace Piers and Foundations**	$34,384,000	1.00%	1.00%	1.00%	1.00%	1.00%	$344,000	$399,000	$462,000	$621,000	$1,507,000	1
3	Bascule Pier Ongoing Rotation and Settlement	**Replace Piers and Foundations**	$34,384,000	2.50%	12.50%	25.00%	50.00%	95.00%	$860,000	$4,983,000	$11,552,000	$31,051,000	$143,199,000	1
4	Bascule Pier Concrete/Reinforcement Deterioration	**Rehabilitate Piers**	$16,143,000	5.00%	30.00%	60.00%	95.00%	100.00%	$807,000	$5,614,000	$13,017,000	$27,698,000	$70,769,000	8
	(i.e. chloride concentration, ASR, thin cover, large cracks)													
5	Concrete Approach Timber Piling Degradation	**Replace Approach Piers and Fndtn's.**	$18,824,000	4.00%	20.00%	50.00%	95.00%	100.00%	$753,000	$4,364,000	$12,649,000	$32,298,000	$82,523,000	1,9
	(similar to 1971 problem)													
6	In-Water Approach Pier and Framing - Concrete/Reinforcement Deterioration	**Replace Approach Piers and Fndtn's.**	$18,824,000	4.00%	20.00%	50.00%	95.00%	100.00%	$753,000	$4,364,000	$12,649,000	$32,298,000	$82,523,000	1,9
	(i.e. chloride concentration, ASR, thin cover)													
7	Other Approach Pier and Framing - Concrete/Reinforcement Deterioration	**Replace Approach Pier and Deck**	$13,470,000	4.00%	20.00%	50.00%	95.00%	100.00%	$539,000	$3,123,000	$9,051,000	$23,112,000	$59,051,000	1
8	Concrete Deck Deterioration	**Deck Reconstruction**	$3,956,000	8.00%	40.00%	60.00%	95.00%	100.00%	$316,000	$1,834,000	$3,190,000	$6,788,000	$17,343,000	1
	(due to freeze/thaw, chloride concentration, etc.)													
	Mechanical Drive Risks													
9	Primary Reducer	**Repair Primary Reducer**	$200,000	10.00%	10.00%	30.00%	50.00%	80.00%	$20,000	$23,000	$81,000	$181,000	$701,000	
10	Differential (one side only)	**Replace Differential**	$356,000	10.00%	10.00%	30.00%	70.00%	100.00%	$36,000	$41,000	$144,000	$450,000	$1,561,000	5
11	Intermediate Reduction (one side only)	**Replace Interm. Reduction**	$1,000,000	10.00%	10.00%	20.00%	30.00%	50.00%	$100,000	$116,000	$269,000	$542,000	$2,192,000	
12	Rack/Pinion (one side only)	**Replace Rack/Pinion**	$1,000,000	0.40%	2.00%	4.00%	8.00%	12.00%	$4,000	$23,000	$54,000	$144,000	$526,000	2
13	Rocker/Track Engagement (one side only)	**Repair Rocker/Track Engagement**	$1,000,000	0.40%	2.00%	4.00%	8.00%	12.00%	$4,000	$23,000	$54,000	$144,000	$526,000	2
	Miscellaneous Mechanical/ Electrical Risks													
14	Electrical System Obsolescence	**Replace Electrical System**	$2,060,000	10.00%	10.00%	30.00%	70.00%	100.00%	$206,000	$239,000	$831,000	$2,604,000	$9,031,000	6
15	Span Seating	**Repair Span Seating**	$4,000,000	0.40%	2.00%	4.00%	8.00%	12.00%	$16,000	$93,000	$215,000	$578,000	$2,104,000	2
	(i.e. tips of leaves, shear keys, tail stops,etc.)													
16	Gate and Signage	**Replace Gates and Signage**	$200,000	10.00%	10.00%	30.00%	70.00%	100.00%	$20,000	$23,000	$81,000	$253,000	$877,000	
17	Ongoing Operation and Maintenance	**Ongoing Cost**	$560,000	100.00%	100.00%	100.00%	100.00%	100.00%	$560,000	$649,000	$753,000	$1,011,000	$2,455,000	4
	Discrete Risks (Subtotal)								**$4,929,000**	**$21,946,000**	**$52,865,000**	**$128,096,000**	**$395,872,000**	9
	'Random' (or Intermittent) Risks													
	SEISMIC (See summation Table 8.1, Ibsen report August 2001)													
18	EQ-20 (10% exceedence in 2 years)	**Repair (without seismic retrofit)**	$500,000	5.00%	25.00%	50.00%	99.00%	99.00%	$25,000	$160,000	$410,000	$1,328,000	$5,841,000	12
19	EQ-50 (10% exceedence in 5-years)	**Repair (without seismic retrofit)**	$820,000	1.00%	10.00%	20.00%	40.00%	90.00%	$8,000	$105,000	$269,000	$880,000	$8,708,000	10,11
20	EQ-95 (10 % exceedence in 10-years)	**Repair (without seismic retrofit)**	$8,204,400	1.00%	5.00%	10.00%	20.00%	55.00%	$82,000	$525,000	$1,344,000	$4,404,000	$53,245,000	7,10
21	EQ-190 (10% exceedence in 20-years)	**Replace Bridge**	$82,044,000	0.40%	2.50%	5.00%	10.00%	25.00%	$328,000	$2,625,000	$6,720,000	$22,019,000	$242,023,000	1,10
22	EQ-475 (10% exceedence in 50-years)	**Replace Bridge**	$82,044,000	0.20%	1.00%	2.00%	4.00%	10.00%	$164,000	$1,050,000	$2,688,000	$8,808,000	$96,809,000	1,10
	Maximum Risk Due to Seismic Event (Subtotal)								**$328,000**	**$2,625,000**	**$6,720,000**	**$22,019,000**	**$242,023,000**	
	Vessel Impact													
23	Bascule	**Repair Bascule Leaves**	$5,738,000	5.00%	5.00%	5.00%	5.00%	5.00%	$287,000	$333,000	$386,000	$518,000	$1,258,000	1
24	Bascule Pier	**Replace Pier and Foundation**	$34,384,000	2.50%	2.50%	2.50%	2.50%	2.50%	$860,000	$997,000	$1,155,000	$1,553,000	$3,768,000	1
25	Approach Piers	**Replace Approach Piers**	$18,824,000	1.00%	1.00%	1.00%	1.00%	1.00%	$188,000	$218,000	$253,000	$340,000	$825,000	1

	Year 0-1	<5 Years	<10 Years	<20 Years	< 50 Years	Notes
TOTAL COST-BASED RISK	**$6,592,000**	**$26,119,000**	**$61,379,000**	**$152,526,000**	**$643,746,000**	
REPLACEMENT COST	**$82,044,000**	**$95,111,000**	**$110,260,000**	**$148,181,000**	**$359,673,000**	1
THRESHOLD COST	**$57,431,000**	**$66,578,000**	**$77,182,000**	**$103,727,000**	**$251,772,000**	3